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OF THE

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OF

CIVIL ENGINEERS

VOLUME 88

1925

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ERRATA

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Pages 1096 and 1098, Equation (23); page 1097, Equation (26); page 1099, Equation (29); page 1143, Equation (59); and page 1145, Equation (60), for term, "0.573 r_s" read "57.3 r_s".

Page 1144, Table 13, Column (G), lines 22 and 23, transpose figures "712" and "1072".

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Page 35, Equation (18), for second term:

$$\text{“} + \left[\frac{C\,k_a\,P_3 - h_a\,k_a\,P_3 + S_1\,k_b\,h_a + P_3\,k_b\,h_a}{h_a\,k_a - k_b\,h_a - C\,k_a}\right]^2\text{”}$$

read,

$$\text{“} + \left[\frac{C\,k_a\,P_3 - h_a\,k_a\,P_3 + S_1\,k_b\,h_a + P_3\,k_b\,h_a}{h_a\,k_a - k_b\,h_a - C\,k_a}\right]^2\text{”}$$

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Paper No. 1551

THE HYDRAULIC DESIGN OF THE SHAFT SPILLWAY FOR THE DAVIS BRIDGE DAM, AND HYDRAULIC TESTS ON WORKING MODELS*

By Ford Kurtz,† M. Am. Soc. C. E.

With Discussion by Messrs. H. B. Muckleston, H. J. F. Gourley, B. F. Jakobsen, Allen Hazen, Karl R. Kennison, H. de B. Parsons, H. F. Dunham, Eugene E. Halmos, Charles W. Comstock, F. W. Scheidenhelm, H. K. Barrows, E. W. Lane, I. Gutmann, and Ford Kurtz.

Synopsis

The object of this paper is to present both the theoretical and the experimental treatments of a new type of shaft spillway for earth dams. Heretofore, shaft spillways seldom have been used and very little information concerning them seems to be available in published form.

The paper treats of the methods of hydraulic design for a large shaft spillway now under construction, the application of these methods to the design of a model of the spillway, and the hydraulic tests on this model, including a discussion of the results of the tests.

On the basis of the data herein submitted, this new type of shaft spillway has been adopted as the most satisfactory and economical construction in the case of a 200-ft. earth dam across a narrow valley, with steeply rising hills on both sides, and it is believed that the data not only justify this particular decision, but also indicate the suitability of the design for any high earth dam similarly located.

General

The Davis Bridge Dam, an earth-fill structure, 200 ft. high, now under construction on the Deerfield River about ¾ mile south of Whitingham, Vt.—a

* Presented at the meeting of March 5, 1924.
† Care, J. G. White Eng. Corporation, New York, N. Y.

station on the Hoosac Tunnel and Wilmington Railroad—will impound water for a new hydro-electric development of the New England Company's power system and, by using part of the stored waters, will also serve to increase the dry-season flow of the river for other hydro-electric plants of the Company located lower down on the Deerfield River. The valley at the site is so narrow and the sides are so steep that the crest of the dam will be about 50 ft. shorter than the extreme base width of approximately 1 300 ft. (Fig. 1.) The dam will contain about 1 850 000 cu. yd. of earth and will create a reservoir 10 miles long and of a capacity of 5 672 000 000 cu. ft. computed for the elevation of the top of the flash-boards, 6 ft. above spillway level. The dam is being constructed by the so-called "modified hydraulic-fill process". A concrete-lined diversion tunnel, 1 528 ft. long, with a net cross-sectional area of 396 sq. ft., has been driven through the earth and rock on the east side of the river to take care of the flow of the stream during the construction period. The downstream part of this tunnel is to be utilized for the spillway. These features of the dam and the tunnel are shown on Fig. 1.

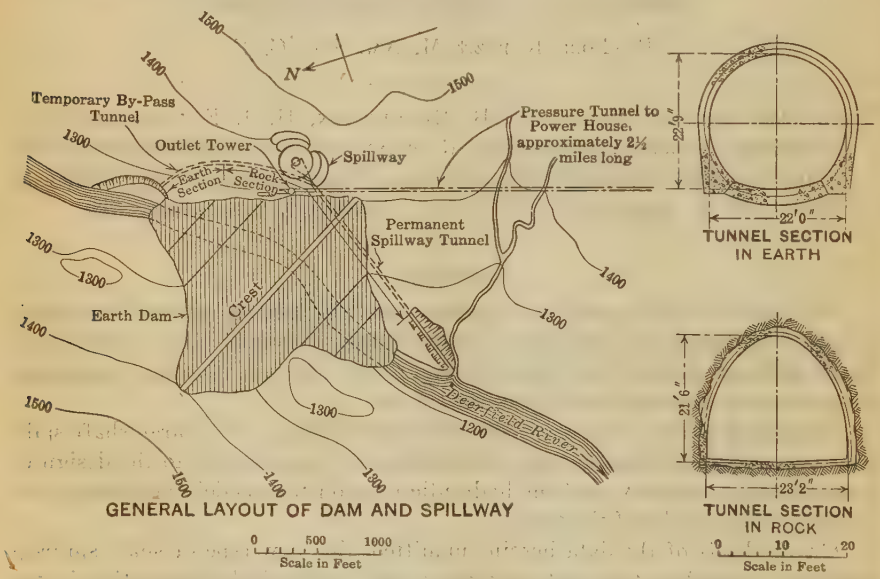

FIG. 1.

The drainage area tributary to the Deerfield River at the dam site is about 184 sq. miles (of which approximately 29.3 sq. miles are also tributary to an existing storage reservoir of the Company on the East Branch of the Deerfield River near Somerset, Vt.). Records of stream flow at several points have been kept by the U. S. Geological Survey for a number of years, and private records have also been kept by the Company at its various plants since their completion. From a study of these data, and after consideration of the effect of the large storage capacity of the reservoir above spillway level in smoothing out the sharp flood peaks, it was decided that the spillway should be designed to have a maximum capacity of 27 000 sec-ft.

At the dam site, the hills on both sides of the river rise to great heights above the crest of the earth dam; at the same time, the lowest point in the range of hills surrounding the reservoir site is more than 100 ft. higher than the crest. The nature of the topography thus prevents the construction of a spillway at a low divide in the surrounding hills—a method which, if feasible, is usually the most economical and satisfactory one for carrying flood waters past an earth dam. Where this method cannot be adopted, it has been customary to construct a masonry overflow spillway at one end of the dam, and the topography usually has been such as to require a spillway channel for conducting the water from the foot of the spillway down the hillside to the bed of the river. In designing and constructing such a channel, every precaution must be taken to protect the earth-fill and the hillside against wash by the flood waters. There are numerous variations of this latter method of design, a common one being a spillway constructed at right angles to, instead of in line with, the crest of the dam and forming a re-entrant angle in the reservoir. In such a design, the spillway channel also projects into the reservoir, as shown in the construction proposed for the Tieton Dam of the U. S. Reclamation Service.*

A re-entrant spillway having a channel on the hillside was first considered for the Davis Bridge Dam. It was found that the hydraulic problems involved in determining the dimensions of the re-entrant section of the spillway channel could be solved only approximately, requiring, as a result, the adoption of ample dimensions, in order to insure the required capacity. Due to the steepness of the hillsides, these ample dimensions would have involved a large amount of excavation for this re-entrant section of the channel, as well as great expense for the construction of the remaining portion for the drop of nearly 200 ft. As a result, the preliminary estimates of cost for this type of spillway were very high and, therefore, it was decided to investigate the possibility of carrying the flood waters through a vertical shaft into the diversion tunnel previously mentioned, thus eliminating the costly hillside channel. This plan involved the placing of a concrete bulkhead across the tunnel at some point between the foot of the shaft and the up-stream portal after the completion of the dam and when the tunnel is not needed any longer for diversion purposes. Estimates based on a preliminary design of this shaft spillway showed that a material saving would be effected by its use. Consequently, it was tentatively adopted, the final decision being contingent on the results of a series of hydraulic tests to be carried out on a working model. The model was built in general to a scale of one-thirty-sixth of the linear dimensions of the full-sized spillway. (Fig. 2.)

On account of the great height of the earth dam and the large quantity of water impounded, it is necessary to have every assurance that the spillway will operate at least as well as the design indicates and especially that it will have the required capacity under the specified head. For these reasons, the satisfactory test of a working model was made a condition precedent to the adoption of the shaft spillway. One objection that has been advanced at times

* This spillway has been described briefly and illustrated in *Engineering News-Record*, December 1, 1921.

against such a spillway is that it would entrain air in such large quantities as to affect seriously its capacity and general operation. It was desired particularly to investigate this condition by capacity tests, inasmuch as any theoretical investigation would be practically impossible.

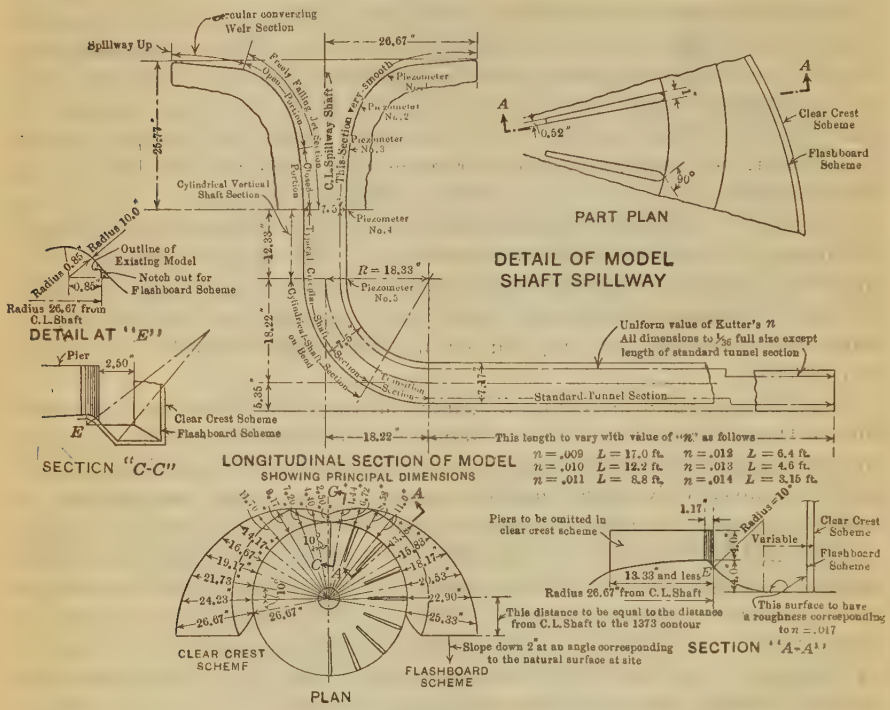
Circular converging Weir Section
Spillway Up
26.67"
Piezometer No.1
Piezometer No.2
Piezometer No.3
Piezometer No.4
Piezometer No.5
C.L. Spillway Shaft
This Section very Smooth
Cylindrical Vertical Shaft Section
Outline of Existing Model
Notch out for Flashboard Scheme
Radius 26.67 from C.L. Shaft
DETAIL AT "E"
Pier
Clear Crest Scheme
Flashboard Scheme
SECTION "C-C"
PART PLAN
DETAIL OF MODEL SHAFT SPILLWAY
Uniform value of Kutter's n
All dimensions to 1/36 full size except length of standard tunnel section
Standard Tunnel Section
This length to vary with value of "n" as follows
n = .009 L = 17.0 ft. n = .012 L = 6.4 ft.
n = .010 L = 12.2 ft. n = .013 L = 4.6 ft.
n = .011 L = 8.8 ft. n = .014 L = 3.15 ft.
LONGITUDINAL SECTION OF MODEL
SHOWING PRINCIPAL DIMENSIONS
Piers to be omitted in clear crest scheme
Variable
13.33 and less
Radius 26.67 from C.L. Shaft
This distance to be equal to the distance from C.L. Shaft to the 1373 contour
This surface to have a roughness corresponding to n = .017
SECTION "A-A"
CLEAR CREST SCHEME
FLASHBOARD SCHEME
Slope down 2" at an angle corresponding to the natural surface at site
PLAN

FIG. 2.

NOTATION

For convenience, all the symbols used in the design have been grouped together in the following list:

Q = flow of water, in cubic feet per second.

Q_1 = flow of water over spillway model, in cubic feet per second, when different from flow, Q, over measuring weir.

C = coefficient in the usual formula for flow over a rectilinear weir, $Q = C\,L\,H^{\frac{3}{2}}$.

L = length of a rectilinear weir, in feet.

H = depth of water on a weir, in feet, measured a sufficient distance up stream from the weir to eliminate the surface drop.

B = crest width of a flat crest, or slightly sloping crest, weir, in feet.

k = coefficient of effective head for a weir, used as a multiplier for gross head, H, to allow for contraction and eddy losses on the weir; abstract number.

g = acceleration due to gravity, in feet per second per second.

r = horizontal distance, in feet, from vertical axis of a circular converging weir to some point on the weir crest.

r_0 = value of r for point on weir at end of "normal acceleration" section.

h = depth of water, in feet, on a circular converging weir at distance, r, from vertical axis.

h_0 = value of h for point on weir at end of "normal acceleration" section.

u = velocity of water, in feet per second, on a circular converging weir at distance, r, from vertical axis.

u_0 = value of u for point on weir at end of "normal acceleration" section.

X = drop in elevation of weir crest between points corresponding to distances, r_0 and r, in feet.

n = Kutter's coefficient of roughness.

h_1 = depth of water, in feet, on a rectilinear flat crest weir after the water surface has become practically horizontal.

z = decrease in depth of water, from value, h_1, at down-stream edge of a flat crest weir, in feet.

x = horizontal distance measured from the down-stream edge of a converging flat crest weir toward the vertical axis of the weir, in feet.

y = vertical distance measured downward from the down-stream edge of a converging flat crest weir, in feet.

a = horizontal distance from the down-stream edge of a converging flat crest weir to the vertex of the parabolic center line of the freely falling jet, measured radially, in feet.

b = vertical distance from the down-stream edge of a converging flat crest weir to the vertex of the parabolic center line of the freely falling jet, in feet.

v_0' = horizontal velocity of the water at the vertex of the parabolic center line of the freely falling jet, in feet per second.

t = time, measured from the instant when a given particle of water passes the vertex of the parabolic center line of the freely falling jet.

c = Hazen and Williams' coefficient in their formula for the flow of water.

c = Chezy coefficient in Chezy formula for the flow of water, $V = c\sqrt{RS}$.

R = hydraulic radius, in feet.

S = slope of hydraulic gradient, abstract number.

l = length of a closed conduit, in feet.

V = velocity in a closed conduit, in feet per second.

D = head on a measuring weir unconnected for velocity of approach, in feet.

h_v = velocity head, in feet.

h' = thickness of the freely falling jet, measured normal to the parabolic center line at a point, x-y, in feet.

v' = velocity of the freely falling jet at a point, x-y, in feet per second.

x_1 = height of "mushroom" of water above the point when the freely falling jet closes in on itself, in feet.

Q_2 = flow which reaches spillway through the spiral approach channel, in cubic feet per second.

V_2 = tangential velocity in spiral approach channel, in feet per second.

Spillway Crests

The first design of the shaft spillway contemplated carrying the maximum flood flow of 27 000 sec-ft., with a depth of water, H, of 7 ft. on the spillway crest which was to be entirely clear, as compared with the final design which has a crest provided with sixteen bridge piers and requires a depth of water on the crest of 8 ft. to discharge the maximum flood flow. The methods used in design were practically the same in both cases. That for the clear crest with a depth of 7 ft. of water will first be described in detail, following which a brief outline of the modifications necessary to adapt it to the type with flashboard piers, having a depth of 8 ft. on the crest, will be given.

First Design—Flat Crest Type

The section of the diversion tunnel which is to serve as a part of the spillway (Fig. 1) has a "horseshoe" cross-section, equivalent in area to a circle 22.5 ft. in diameter. It was decided to make the vertical spillway shaft circular in cross-section, to reduce it to a uniform diameter of 22.5 ft. as soon as the water falling over the spillway lip could be accelerated to the required velocity, and to connect this cylindrical shaft to the tunnel by a 90° bend, a part of which would be circular in cross-section and the remainder would consist of transition from circular to "horseshoe" shape. Experimental data on the excess loss of head due to bends show that of all bends having a radius of less than 10 or 12 diameters the one having a radius of 3.5 to 4 diameters causes the least total excess loss of head. In this case, the practical difficulties in the construction of such a vertical bend compelled the adoption of a radius of about 2.5 diameters, which shape probably causes an excess loss 20% greater than the minimum.

Having fixed the foregoing features of the shaft and its connection to the tunnel, the next step was to design the spillway lip and the converging section of the shaft connecting it with the cylindrical section. The velocity in the latter section at full capacity being 68 ft. per sec., it is evident that the water must have a free, frictionless fall of more than 72 ft., in order to attain it, and the time required to do so would be about 2.12 sec. Inasmuch as the water must have an initial horizontal velocity when it passes the spillway lip, it will take a parabolic course in its free fall. If this free fall should begin at the spillway lip, the initial horizontal velocity with a 7-ft. head could not possibly exceed 17.5 ft. per sec., with the most favorable design of lip, and the discharge per linear foot could not possibly exceed 74 cu. ft. per sec. At this rate for

the total discharge, a circular spillway lip with a radius of 58 ft. would be required and the average time for the water to cover the horizontal distance from the lip to the cylindrical shaft would be 3.00 sec. This disagreement between the intervals of time required for horizontal and vertical travel of the water shows: First, that the free fall cannot begin at the spillway lip, inasmuch as the jet would have acquired its final vertical velocity before it had traveled all the way from the lip to the cylindrical shaft; and, second, that because free fall cannot begin at the spillway lip, that lip must be in the nature of a flat crest weir rather than a typical parabolic spillway section of high discharge coefficient.

General Features.—The general type of design decided on for detailed study was made up of the following distinct parts (Fig. 3):

1.—A circular converging weir section with a slightly rounded up-stream edge and having a discharge coefficient equal to that of a rectilinear flat crest weir with a slightly rounded up-stream edge; this equality of coefficients to be obtained by giving to the circular converging weir crest a downward slope just sufficient to compensate for the convergence of the weir, that is, to compensate for the decrease in length of weir as the water approaches the center of the circular weir.

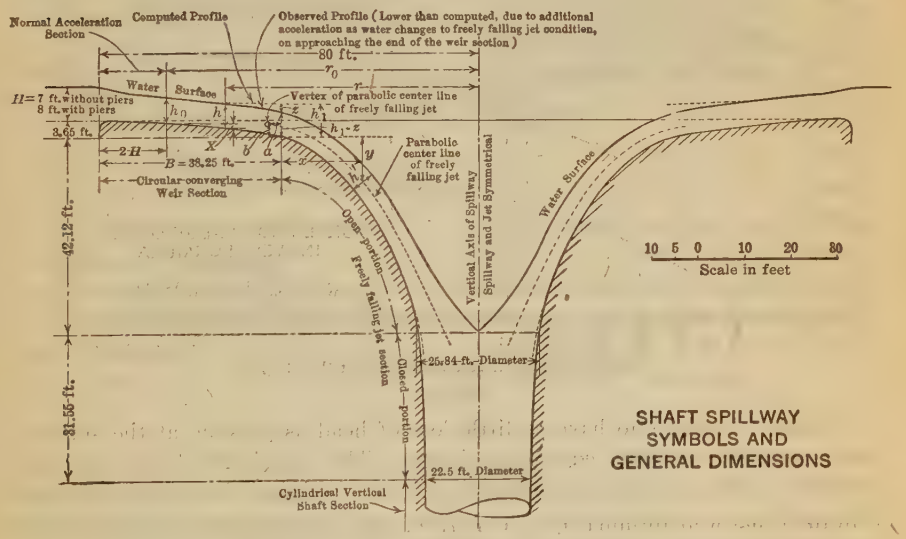

FIG. 3.

2.—An open portion of a freely falling jet section, beginning at the down-stream edge of the circular converging weir section; the surface to follow the lower nappe of the freely falling converging circular jet and the section to continue downward until the jet "closes in" on itself.

3.—A closed portion of a freely falling jet section, beginning at the lower end of the open portion and continuing downward to

the elevation at which the water has acquired a vertical velocity equal to the final velocity required throughout the length of the closed sections of spillway conduit.

4.—A closed cylindrical vertical shaft section.

5.—A closed section of circular cross-section and located on bend.

6.—A closed section on bend, forming a transition from the circular cross-section to the standard tunnel cross-section.

7.—A closed, straight, standard tunnel section, practically horizontal, already designed as a part of the by-pass tunnel for use during the construction of the dam.

Having arrived at these decisions as to general features of the design, the determination of the actual details was a matter of "cut and try", a fact that must be borne in mind throughout the remainder of the description of design methods, to explain how it is possible at times to use certain features and dimensions which are not determined until a later stage of the design. By referring to Fig. 3 which shows the final shape, the various steps in the design may be followed readily.

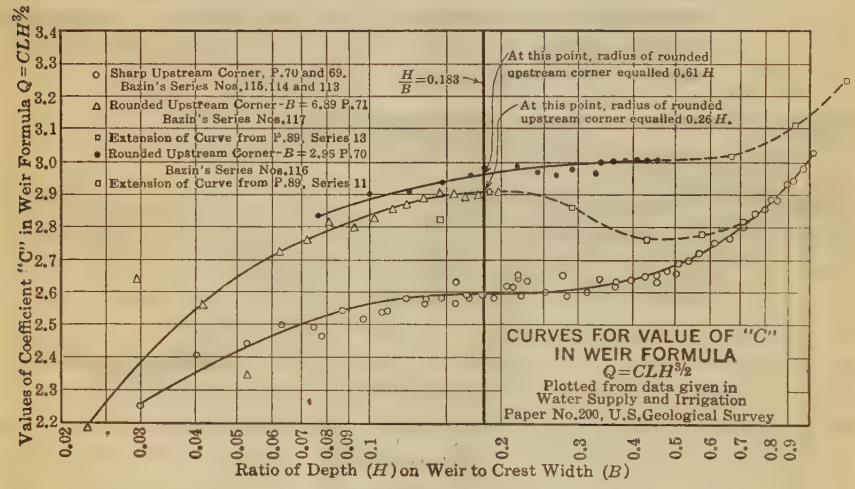
Values of Coefficient "C" in Weir Formula $Q=CLH^{3/2}$
Ratio of Depth (H) on Weir to Crest Width (B)
Sharp Upstream Corner, P.70 and 69. Bazin's Series Nos.115, 114 and 113
Rounded Upstream Corner-$B=6.89$ P.71 Bazin's Series Nos.117
Extension of Curve from P.89, Series 13
Rounded Upstream Corner-$B=2.95$ P.70 Bazin's Series Nos.116
Extension of Curve from P.89, Series 11
$\frac{H}{B}=0.183$
At this point, radius of rounded upstream corner equalled 0.61 H
At this point, radius of rounded upstream corner equalled 0.26 H.
CURVES FOR VALUE OF "C" IN WEIR FORMULA $Q=CLH^{3/2}$
Plotted from data given in Water Supply and Irrigation Paper No.200, U.S.Geological Survey

FIG. 4.

Design.—In order to have as little loss of head as possible at the spillway lip, a rounded up-stream edge was adopted. The experimental data used for fixing the radius of this rounded edge and for obtaining the proper value for C in the usual weir formula, $Q = C\,L\,H^{\frac{3}{2}}$, were those of Bazin and the U. S. Deep Waterways Board.* The experimental values of C were plotted against corresponding values of the ratio of head on the weir (H) to total width of weir crest (B). The resulting curves are shown on Fig. 4. It will develop later that the width of flat crest adopted for the Davis Bridge Spillway is 38.25 ft. and as the original specified head on the weir at maximum discharge was 7 ft., the ratio of head, H, to crest width, B, under that condition would be 0.183. A reference to Fig. 4, shows the experimental value of C corresponding

* *Water Supply Paper No. 200*, U. S. Geological Survey, pp. 69–71 and 89.

to $\frac{H}{B} = 0.183$ to be 2.91 for a flat crest weir, when the radius of the rounded up-stream corner equals 0.26 of the head on the weir, and 2.96 when the radius of the rounded corner equals 0.61 of the head on the weir. The radius adopted for the up-stream corner was 2.5 ft. (corresponding to 0.36 of the head), and the value of C was taken as 2.90.

Three of the four variables in the weir formula, $Q = C\,L\,H^{\frac{3}{2}}$, having been fixed, the fourth, which is the length of the clear crest, L, can be found at once to be 502 ft. for $H = 7$ ft. For a circular flat crest weir, the corresponding radius of the up-stream edge is 80 ft.

According to the "principle of least energy", as enunciated by Unwin, and independently also by Frizell, the flow over a flat crest weir, if it takes place without eddies at the up-stream corner and without frictional resistances, follows the law, $Q = 3.087\ L\ H^{\frac{3}{2}}$. If it is assumed that a loss of head equal to $(1 - k)\ H$ occurs at the up-stream corner due to eddies, the principle previously mentioned leads to the formula, $Q = 3.087\ L\ (k\ H)^{\frac{3}{2}}$, k being the coefficient of effective head. From this formula, it is possible to compute the loss of head at the up-stream corner due to eddies if C is known for any given set of conditions, because $k^{\frac{3}{2}} = \frac{C}{3.087}$. In this case, where C is assumed as 2.90, the value of k is 0.959, that is, the loss of head is 0.041 H, or 0.3 ft., when $H = 7$ ft. Under these conditions, the depth of water on a rectilinear weir of this type after the water surface becomes practically horizontal over the flat crest, would be $\frac{2}{3}\ k\ H$, or 4.48 ft., whereas the velocity would be $\sqrt{2g\left(\frac{1}{3}\ k\,H\right)}$, or 12.00 ft. per sec.

The acceleration of the water as it passes over a rectilinear flat crest weir continues for a distance from 1.5 H to 2 H beyond the up-stream corner, which, in the present case, would mean for a distance from 10.5 to 14 ft.* This general statement is based on the profiles of water surfaces taken by the U. S. Deep Waterways Board in the course of its experiments on the flow of water over various types of weirs made at the Hydraulic Laboratory of Cornell University. From an examination of the plotted profiles,† the distance seems to be at least 2 H for those cases where the ratio, $\frac{H}{B}$, is small enough to show definitely the distance over which acceleration takes place. If 14 ft. is adopted as the distance, then acceleration continues to a point where the radius, r, measured from the center of the shaft, is 66 ft. At this point, the length of weir crest, that is, the circumference of the circle of radius 66 ft., is less than the length of the up-stream edge, or weir lip, and, therefore, the depth of 4.48 ft. and the velocity of 12.00 ft. per sec., which were determined on the basis of a rectilinear or non-converging crest, cannot obtain under the actual conditions.

* *Water Supply Paper No. 200*, U. S. Geological Survey, p. 111.
† *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 329.

In order to comply with the condition of a continual decrease in the length of crest as the water approaches the center of the spillway and shaft, and also to maintain the same discharge as would obtain over a rectilinear flat crest weir of length equal to the outer circumference of the circular converging weir, evidently either the depth or the velocity of the water, or both, must increase above the values obtaining under the condition of constant length of crest. The following theoretical treatment of this phase of the problem was used in the design.

If, at the circumference of a circle of radius, r_0, the water has a depth, h_0, and is flowing horizontally (vertical motion so slight as to be disregarded), and radially inward at a velocity, u_0, and if, for a circle of radius, r, the corresponding depth and velocity are designated as h and u, then, for steady flow:

$$Q = 2\,\pi\,r_0\,h_0\,u_0 = 2\,\pi\,r\,h\,u$$

The change in velocity head between these two points equals

$$\frac{1}{2\,g}\left(\frac{Q^2}{4\,\pi^2\,r^2\,h^2} - {u_0}^2\right)$$

If friction is neglected this change in velocity head represents the necessary drop in the elevation of water surface between the two circles of radius, r_0 and r. It follows that the drop in the weir crest between the two circles must be:

$$h + \frac{1}{2\,g}\left(\frac{Q^2}{4\,\pi^2\,r^2\,h^2} - {u_0}^2\right) - h_0 = X$$

The best value of h for economy is that which will require the least drop in the weir crest, that is, which will make X a minimum. This value of h is determined by letting $\frac{d\,X}{d\,h} = 0$, or,

$$1 - \frac{Q^2}{4\,\pi^2\,r^2\,g\,h^3} = 0$$

From this equation the following relations are obtained:

$$h = \sqrt[3]{\frac{Q^2}{4\,\pi^2\,r^2\,g}}, \quad \text{and,} \quad u = \frac{Q}{2\,\pi\,r\,h} = \sqrt[3]{\frac{Q\,g}{2\,\pi\,r}}$$

also, $\frac{h}{2} = \frac{u^2}{2\,g}$ = velocity head at any point.

The foregoing treatment shows that both the depth and the velocity vary inversely with the radius for best results, or that both h and u must increase as the water flows toward the center. From the equations, as given, the proper values of h and u for the point at which acceleration corresponding to a rectilinear weir crest has ceased, namely, for $r = 66$ ft., are $h_0 = 5.08$ ft. and $u_0 = 12.76$ ft. per sec. These values are in contrast with $h_0 = 4.48$ ft. and $u_0 = 12.00$ ft., per sec., computed for the rectilinear weir crest. Using a value of $n = 0.014$ in Kutter's formula, and taking the average hydraulic radius as about 6 ft. and the average velocity as about 10 ft. per sec., it can be shown easily that the friction in this 14-ft. section over which "normal acceleration" takes place does not exceed 0.01 ft., which is negligible. Hence, the total drop

in water surface from reservoir level to the circle of radius 66 ft., is 0.3 ft. entrance loss plus the velocity head of 2.54 ft. (corresponding to a velocity of 12.76 ft. per sec.), or a total of 2.84 ft. The water being 5.08 ft. deep at this point, the crest of the weir must be 7.92 ft. below reservoir level, or, in other words, the weir crest must drop 0.92 ft. in the 14-ft. width of crest over which "normal acceleration" takes place.

There is no theoretical method of plotting the water surface for the "normal acceleration" section, but its general shape was predicted from the experimental data on profiles of water surface already mentioned. Beyond this section, the complete profiles of both the water surface and the weir crest can be plotted by the use of the formulas for h and u. Just as friction in the "normal acceleration" section was neglected, so the friction for the remainder of the circular converging weir section was found to be so small that it also was neglected.

It is evident that at some particular value of radius, r, the slightly sloping weir crest should end and the inwardly flowing water allowed to fall freely as a converging circular jet of parabolic vertical section. The down-stream face of the weir crest down to its junction with the cylindrical shaft must conform, of course, to the shape of the lower nappe of this converging jet. The down-stream edge of the weir crest was chosen so that the jet would not close in on itself (at the top of the cylindrical shaft) at such a sharp angle as to produce excessive disturbance due to the loss of its entire horizontal velocity. In order to treat this part of the problem, it was necessary to have a formula for the shape of a jet of water passing over a flat crest weir. Bazin made exhaustive experimental studies of the shape of jets passing various types of weirs, but not of a flat crest weir. The only available data for this type, as already mentioned, are those for the profile of upper nappe only, made by the U. S. Deep Waterways Board. A study of these data, however, was preceded by a theoretical investigation.

If H represents the total head on a rectilinear flat crest weir, h_1, the depth of water on the weir crest after the water surface has become practically horizontal, and $(h_1 - z)$, the depth of water on the down-stream edge of the weir, then, if $(1 - k)\ H$ represents the loss of head at the up-stream edge of the weir,

$$h_1 = \frac{2}{3}\ (k\ H)$$

$$Q = 3.087\ (k\ H)^{\frac{3}{2}}$$

Velocity of upper filaments of jet as it leaves the down-stream edge of the crest

$$= \sqrt{2\ g\left(\frac{1}{3}\ k\ H + z\right)}$$

Velocity of lower filaments of jet as it leaves the down-stream edge of the crest

$$= \sqrt{2\ g\ (k\ H)}$$

Average velocity of jet (approximate)

$$= \frac{\sqrt{2\,g\,(k\,H)} + \sqrt{2\,g\left(\frac{1}{3}\,k\,H + z\right)}}{2}$$

Depth of jet

$$= h_1 - z = \frac{2}{3}\,(k\,H) - z$$

Then,

$$Q = \left[\frac{\sqrt{2\,g\,(k\,H)} + \sqrt{2\,g\left(\frac{1}{3}\,k\,H + z\right)}}{2}\right]\left[\frac{2}{3}\,(k\,H) - z\right] = 3.087\,(k\,H)^{\frac{3}{2}}$$

$$\left(1 + \sqrt{\frac{1}{3} + \frac{z}{k\,H}}\right)\left(\frac{2}{3} - \frac{z}{k\,H}\right) = 0.7698$$

$$\frac{z}{k\,H} = 0.2267$$

$$\frac{h_1 - z}{k\,H} = 0.44$$

This value of $\frac{h_1 - z}{k\,H}$ agrees very well with experimental data.

Velocity of upper filaments $= 0.75\,\sqrt{2\,g\,(k\,H)}$

Velocity of lower filaments $= \sqrt{2\,g\,(k\,H)}$

Average velocity of filaments $= 0.875\,\sqrt{2\,g\,(k\,H)}$

Average head of filaments $= \frac{\left(0.875\,\sqrt{2\,g\,(k\,H)}\right)^2}{2\,g} = 0.766\,(k\,H)$

Because of the surface curve of the jet of water as it approaches the downstream edge of the flat crest weir, the vertex of that parabola which represents the center line of the jet will not be immediately over the edge of the weir. The unknown distances of the vertex back from and above the edge will be denoted by a and b, respectively, whereas the horizontal distances from the edge to the center line of jet, measured toward the center of the shaft, and the vertical distances measured from the edge downward, will be denoted by $+\,x$ and $+\,y$, respectively. If the velocity of the jet at the distance, a, back from the edge is denoted by v_0', the path of a particle of water on the center line of the jet is completely represented by the following equations, in which t represents time measured from the instant when the particle passes the point $(-\,a, -\,b)$:

$$x + a = v_0'\,t;\ \text{and},\ y + b = \tfrac{1}{2}\,g\,t^2$$

Combining these equations,

$$y + b = \frac{(x + a)^2}{\frac{2\,v_0'^2}{g}}$$

Assuming that a and b are small so that v'_0 can be taken as only slightly less than the average velocity of the jet at the edge of the weir and b can be taken as $\left(k\,H - \frac{v'^2_0}{2\,g}\right)$, the equation becomes:

$$y + 0.24\,k\,H = \frac{(x+a)^2}{3.04\,k\,H}$$

When this point had been reached in the theoretical investigation, the experimental data were used to check the correctness of the formula and to obtain the most probable value for a. The value adopted as best fitting the experiments applicable to the present problem was $0.24\,k\,H$, the same as the value of b. The complete formula then becomes:

$$y + 0.24\,k\,H = \frac{(x + 0.24\,k\,H)^2}{3.04\,k\,H}$$

This formula gives $y = -0.22\,k\,H$ for $x = 0$, which is correct, as the thickness of the jet at that point equals $0.44\,k\,H$. Assuming no internal pressure in the jet, the resultant velocity, v', at any point of the center line of the jet is,

$$v' = \sqrt{v'^2_0 + 2\,g\,(y+b)} = \sqrt{2\,g\,(y + k\,H)}$$

This is not exactly true for the portion of the jet up stream from the edge of the weir, as internal pressures do exist there, and, in fact, were assumed in the derivation of the formulas for h and u for the circular converging weir section. The formula for v', however, is valid for the free portion of the jet, down stream from the edge of the weir.

As $k\,H = \frac{3}{2}\,h_1$ for a rectilinear crest, the equations may also be written as follows:

$$(y + 0.36\,h_1) = \frac{(x + 0.36\,h_1)^2}{4.56\,h_1}$$

and,

$$v = \sqrt{2\,g\,(y + 1.5\,h_1)}$$

For the circular converging weir, h_1 does not have a single value as in the case of the rectilinear flat crest weir, but it was decided that in adapting the foregoing formula to the converging weir, h_1 should be taken as the depth determined by the formula, $\left(h = \sqrt[3]{\frac{Q^2}{4\,\pi^2\,r^2\,g}}\right)$, for the value of r corresponding to the down-stream edge of the slightly sloping weir crest. After a number of trials, the value of r finally adopted for this edge was 41.75 ft. As the radius of the up-stream edge is 80 ft., the width, B, of the flat crest weir, is 38.25 ft., as already noted. The corresponding value of h_1 is 6.90 ft. The equation of the center line of jet is,

$$y + 2.48 = \frac{[\,(41.75 - r) + 2.48]^2}{31.46}$$

The velocity equation is,

$$v' = \sqrt{2\,g\,(y + 10.35)}$$

The down-stream edge of the flat crest weir is 6.90 (depth) + 3.45 (velocity head) + 0.30 (entrance loss) = 10.65 ft. below the water surface of the reservoir.

The velocity, v', and radius, r, of the center line of the jet being known, the thickness of the jet, that is, the depth of water, h', normal to the center line, can at once be found by the formula, $h' = \frac{Q}{2 \pi r v'}$. Points plotted on both sides of the center line, and at a distance of $\frac{h'}{2}$ normally from it, determine the surface of the upper and lower nappes of the jet. It should be noted that the jet increases in thickness as it falls, the increased velocity of the water being more than offset by the rapid convergence of the jet. From the plotted curve of the lower nappe of the jet, which fixes the curve of the down-stream face of the flat crest weir, that is, the converging throat section, the radii of the wetted perimeter of the various sections were measured; the hydraulic radii were then computed and friction allowed for on the basis of Kutter's formula with $n = 0.014$. Corrected velocities of jet and corrected depths of jet were then computed for the final determination of the upper and lower nappes.

For the chosen value of H, the upper nappe of the converging jet closes in on itself at a point about 52.77 ft. below the water surface in the reservoir. It was assumed that, at this point, the entire horizontal velocity of the jet plus a small part of the vertical velocity suddenly is lost by impact and eddies. The residual (vertical) velocity at that point is, therefore, that due to a head of 52.77 ft. less the sum of 0.3 ft., loss of head at entrance, 10.35 ft. of lost velocity head, and 1.04 ft. of friction head in the open portion of the freely falling jet section. The total deductions amount to 11.69 ft., leaving a net head of 41.08 ft., which corresponds to a velocity of 51.4 ft. per sec. The required area of the circular shaft at this level is, therefore, $\frac{27\,000}{51.4}$, or 527 sq. ft., which corresponds to a shaft radius of 12.92 ft. From this point downward, for 31.55 ft., the computations for diameter of shaft, taking friction into account, were made at 10-ft. intervals, the flow being accelerated and the shaft diameter reduced as much as the available head, less friction, would permit. At the lowest point mentioned, which is 84.32 ft. below the reservoir level, the final velocity of 67.90 ft. per sec. is attained, and the top of the cylindrical section of shaft is reached.

The hydraulics of the cylindrical shaft and tunnel are, of course, very simple. There is a straight cylindrical section, 37 ft. long, a curved section, 86 ft. long, including the transition from the circular cross-section to the horseshoe tunnel section, and 832 ft. of straight tunnel section at the end of which is an open channel about 325 ft. long, discharging into the river below the dam site. Assuming that the water level in the open channel is at the crown of the tunnel at the discharge end, the available head required to overcome friction and the excess losses in the bend and in the transition, is about 80 ft., measured from the elevation at which the water attains its final velocity of 67.9 ft. per

sec. to the water surface in the open channel. If n in Kutter's formula is taken as 0.014 (equivalent to taking $c = 93.9$ in the Hazen and Williams formula), and if the excess losses in the bend and in the transition are taken as 35% and 5%, respectively, of the velocity head, the total loss of head for a flow of 27 000 sec-ft. is about 73 ft. It was to be expected, therefore, that the tunnel would be capable of discharging more than 27 000 sec-ft. with the head available; tests on the model, described later, bore out this expectation.

In the design of the clear crest type for $H = 7$ ft., no allowance was made for the loss of head in the spiral approach channel, nor for any part of the velocity head corresponding to the spiral or tangential velocity of the water when it reaches the spillway lip and begins its distinctly radial movement toward the center of the shaft. In other words, the first design was made as if the spillway was to be built in the center of a large lake with deep water on all sides, and the setting for the first tests on the working model was an attempt to simulate this condition. The actual location of the spillway on the side-hill is such that the approach to approximately one-half of it is from deep water; the other half, however, is approached by means of a spiral channel. The depth of this channel was fixed at 12 ft. below the spillway crest, which compares favorably with a depth of 13.65 ft. found by adopting the same ratio of depth of channel to head on weir, as obtained in the Bazin experiments, Series 117, for the condition, $\frac{H}{B} = 0.183$, at which time $C = 2.91$. It was realized, of course, that there is a great difference between a straight and a spiral approach channel, but no attempt was made to evaluate this in the first design. The spiral approach channel arbitrarily was assumed to have a tangential velocity of 5.4 ft. per sec., and its dimensions were determined on the basis of this velocity and the depth previously mentioned. The curve joining the spillway lip and the level bottom of the approach channel was adopted on the basis of other published information.* The neglect of these losses in the spiral approach channel undoubtedly accounts in considerable measure for the differences found between the experiments with, and without, such channel. In the design of the pier crest type of spillway for $H = 8$ ft., ample allowance was made for losses, as will be described subsequently, and the predicted capacity of the model was almost in exact agreement with the experimental results. A summary of the computations for the clear crest type of spillway will be found in Table 1.

Final Design—Pier Crest Type

In the final design, full allowance was made not only for friction in the spiral approach channel, but also for losses due to the spiral or tangential velocity of the water. A coefficient of contraction was also applied to the weir constant, C, to take care of the contraction at the sixteen radial bridge piers.

The depth of the channel below the spillway crest was made 12 ft., as in the first design, and the tangential velocity in the channel was assumed to be uniform at 5.4 ft. per sec. In computing frictional losses, the value

* *Water Supply Paper No. 200*, U. S. Geological Survey, Fig. 3, p. 16.

TABLE 1.—First Design Computations, Clear Crest Type.

(Elevation of Water in Reservoir, 1 394.0).

Circular Converging Weir Section.

r.	h.	u.	h_v.	Elevation of floor, in feet.	Elevation of water surface, in feet.
80	5.55	9.65	1.45	1 387.	1 392.55
66	5.08	12.76	2.54	1 386.08	1 391.16
60	5.41	13.20	2.70	1 385.59	1 391.00
50	6.12	14.05	3.06	1 384.52	1 390.64
41.75	6.90	(14.92)	3.45	1 383.35	1 390.25

Freely Falling Jet Section: (*a*) Open Portion.

x.	y.	Elevation of center line of jet, in feet.	v'.	$h' = \frac{Q}{2\pi x v'}$	Radius of wetted perimeter, in feet.	R, hydraulic radius.	S.	l.	Partial friction loss, in feet.	Accumulated friction loss, in feet.	Corrected v'.	Corrected h'.
Vertex 44.23	—2.48	1 385.83	22.45	....	...	....	.....	...	.....	....	22.45	....
41.75	—2.28	1 385.63	22.75	4.52	42.2	4.47	0.0062	...	.....	....	22.75	4.52
36.75	—0.70	1 384.05	24.90	4.70	37.7	4.57	0.0072	4.7	0.032	0.03	24.86	4.69
31.75	2.46	1 380.89	28.70	4.71	33.0	4.53	0.0097	5.4	0.046	0.08	28.55	4.74
26.75	7.22	1 376.13	33.55	4.79	28.6	4.47	0.0134	6.5	0.075	0.15	33.25	4.82
21.75	13.59	1 369.76	39.20	5.04	23.9	4.58	0.0179	7.8	0.122	0.28	39.00	5.06
19.75	16.59	1 366.76	41.55	5.24	22.1	4.67	0.0198	3.5	0.066	0.34	41.40	5.26
17.75	19.82	1 363.53	44.05	5.49	20.2	4.82	0.0215	3.7	0.077	0.42	43.70	5.53
15.75	23.30	1 360.05	46.50	5.86	18.4	5.01	0.0231	3.9	0.087	0.50	46.15	5.90
13.75	27.06	1 356.29	48.95	6.38	16.6	5.29	0.0241	4.2	0.099	0.60	48.55	6.43
11.75	31.04	1 352.31	51.50	7.10	15.0	5.55	0.0255	4.4	0.109	0.71	51.10	7.14
9.75	35.30	1 348.05	54.20	8.09	13.5	5.86	0.0269	4.7	0.123	0.84	53.60	8.19
7.75	39.82	1 343.53	56.85	9.72	12.3	6.15	0.0280	4.9	0.135	0.97	56.20	9.84
6.75	42.12	1 341.23	58.05	10.93	11.7	6.32	0.0284	2.5	0.070	1.04	57.50	11.05

Freely Falling Jet Section: (*b*) Closed Portion.

Assume that the entire horizontal velocity plus a small portion of the vertical velocity is suddenly lost at Elevation 1 341.23, where the jet closes in on itself. Then, vertical velocity $= 8.025\sqrt{42.12 - 1.04} = 51.4$ ft. per sec., and radius of shaft = 12.92 ft. Hydraulic radius = 6.46 ft.; $C = 139$; $S = 0.0212$.

At Elevation 1 332, $v' = 8.025\sqrt{50.31} = 56.9$ (approx.); $R = 12.29$; $S = 0.0272$. Friction loss = 0.22; revised $v' = 56.8$; $R = 12\ 30$ ft.

At Elevation 1 322, $v' = 8.025\sqrt{60.09 - 0.30} = 61.95$ (approx.); $R = 11.77$ ft.; $S = 0.0344$.
Friction loss = 0.31, which checks the foregoing assumed value.

At Elevation 1 312, $v' = 8.025\sqrt{59.78 + 10 - 0.40} = 66.80$ (approx.); $R = 11.34$ ft.; $S = 0.0420$.
Friction loss = 0.38, which checks the foregoing assumed value.

At Elevation 1 309.68, $v' = 8.025\sqrt{69.38 + 2\ 32 - 0.10} = 67.90$; $R = 11.25$ ft.; $S = 0.0436$.
Friction loss = 0.10, which checks the foregoing assumed value.

of n in Kutter's formula was taken as 0.035. It was further assumed that the entire velocity head corresponding to the tangential velocity in the spiral approach channel would be lost at the spillway lip and that, in addition to these two losses, there would be a further loss at the piers due to eddies resulting from the tangential velocity in the spiral. These losses* are summarized, as follows:

Entire tangential velocity head lost.	0.45 ft.
Average loss due to friction.....	0.03 "
Head lost at piers due to tangential velocity taken as two-thirds of the tangential velocity head.....	0.30 "
Total	0.78 ft. for half of spillway
Equivalent to.....	0.39 " " entire "

Adopted equivalent net head = 8 — 0.39 = 7.61 ft.

Assuming that there are 16 bridge piers with a clear crest length of 28.25 ft. between piers, and that the end contraction at each pier equals 0.04 H, the end contraction per span amounts to 0.61 ft., which corresponds to a coefficient of end contraction of 0.978. This changes the value of C from 2.91 to 2.845. The entire loss of head due to contraction both on the bottom and at piers can now be determined from the new value of C, and amounts to 5.4%, or 0.41 ft. The necessary length of spillway, L, to pass 27 000 sec-ft., with the gross head of 8 ft., after deducting all the foregoing losses, is found to be 452 ft., which corresponds to 16 spans of 28.25 ft. each. The thickness adopted for each pier at its outer end being 3 ft., the total required circumference of spillway is 500 ft., corresponding to a radius of 79.4 ft. This is practically the same radius as that adopted for the clear crest spillway in the first design. If the crest were rectilinear, the depth of water after the water surface becomes practically horizontal would be $\frac{2}{3} k H$ (in which, $H = 7.61$ ft. and $k = 0.946$), or 4.8 ft., whereas, the velocity would be $\sqrt{2 g \left(\frac{1}{3} k H\right)}$, or 12.43 ft. per sec.

In adapting to the case of the spillway with bridge piers the formulas for h and u previously used for the circular converging weir section after "normal acceleration" is completed, it is evident that the variable length of crest, $2 \pi r$, previously used, should be replaced by $\frac{28.25}{31.25} (2 \pi r)$, assuming the bridge piers to have radial faces and to extend to the theoretical down-stream edge of the flat crest weir, at which point the free fall of the jet begins. With this single change, all the formulas for the first design become applicable to the final design and from them was computed the profile of the converging throat of the spillway down to a junction with the cylindrical shaft. The value adopted for B, the theoretical width of flat crest, was 42 ft.

* For one-half of perimeter only, the other half being open.

The differences between the first and final spillway designs, both as to diameter of spillway lip and shape of the profile, were so slight that the values of the first design were ultimately adopted, except for a slight modification of profile near the spillway lip, in the form of a flat annular space which makes it possible to have a straight line of flash-boards between adjacent piers and, at the same time, have the bottoms of the boards level.

Design of Model

The size of the model was influenced both by the available water supply at places where it was to be tested and by considerations of cost. It was decided to build the model in general to $\frac{1}{36}$ of the linear dimensions of the full-sized spillway, with only such distortions from that scale as might be necessary, in order to maintain a fixed ratio between the flow through the full-sized spillway and the flow through all parts of the model. For this purpose, the following study for the design of the model was made.

1.—Flow over Flat Crest Weir.—If the spillway lip is an exact linear model of the full-sized spillway, the ratio, $\frac{H}{B}$, remains constant if the experimental heads are $\frac{1}{36}$ of the actual heads, and, hence, values of C can be assumed to remain constant. Therefore, the flow ratio will depend on the respective values of $L H^{\frac{3}{2}}$ and will equal $\frac{1}{36}\left(\frac{1}{36}\right)^{\frac{3}{2}}$, or $\frac{1}{7\,776}$. Velocities will obviously vary with $H^{\frac{1}{2}}$, that is, the velocity ratio will be $\frac{1}{6}$.

2.—Flow over Circular Converging Weir Section.—The formulas already derived for the relation between Q, h, and u for this section show that Q varies as $(h^3 r^2)^{\frac{1}{2}}$ and v varies as $\frac{Q}{hr}$. If the flow ratio is $\frac{1}{7\,776}$, and the dimension, r, follows the model ratio, $\frac{1}{36}$, the depth ratio will be $\frac{1}{36}$, the same as the model ratio, and the velocity ratio will be $\frac{1}{6}$. It is seen, therefore, that there need be no dimensional distortion in this section.

The matter of friction ratio for this section still remains to be investigated. Assuming for the full-sized spillway a coefficient of roughness, n, in Kutter's formula of 0.014 and an average hydraulic radius of about 4 ft. for the maximum flow of 27 000 sec-ft., the corresponding value of c in the Chezy formula is 132. The slope of the energy gradient for this section of spillway varies as $\frac{u^2}{c^2 R}$ and should be the same in the model as in the full-sized spillway, as this slope is a ratio or abstract number. Inasmuch as the ratio, $\frac{u^2}{R}$, remains constant as has been noted, it is evident that the energy gradient can remain constant

only if c is the same in both the model and in the full-sized spillway. As c depends on the hydraulic radius (the slope of energy gradient remaining constant), and as the hydraulic radius for the model is only $\frac{1}{36}$ of that for the full-sized spillway, it is obvious that c can only be made the same for both, by changing or "distorting" the value of n in Kutter's formula for the model and that even then c can be made the same only for some particular discharge through the spillway. If the chosen discharge is the maximum flow of 27 000 sec-ft., the hydraulic radius for the model is 0.11 ft. and, for $n = 0.009$, the value of c is 112. This is the lowest value of n given by Kutter so that, in order even to approach frictional similarity, it was necessary to make this part of the model as smooth as practicable. Fortunately, the influence of friction on the design is so slight that the impossibility of simulating conditions, as between the model and the full-sized spillway, is of minor moment.

3.—Freely Falling Jet Section.—The formula for velocity in this section shows that it varies as $\sqrt{y + kH}$. It has already been seen that H for the model is $\frac{1}{36}$ of the head for the full-sized spillway; y is, of course, a linear dimension of the spillway, for which the ratio is also $\frac{1}{36}$. The velocity ratio for this section is, therefore, $\frac{1}{6}$. If the flow ratio is $\frac{1}{7\,776}$, it follows from the formula, $\left(h' = \frac{Q}{2\pi r v'}\right)$, that the depth ratio will be $\frac{1}{36}$, the same as the model ratio. Therefore, no dimensional distortion is necessary for this section.

As far as the friction ratio is concerned, it was found that, for the maximum flow of 27 000 sec-ft., the hydraulic radius of the full-sized spillway varies from 4.47 to 6.32 ft., and that, for a value of $n = 0.014$ in Kutter's formula, the value of c in the Chezy formula is 137. The hydraulic radius for the model varies from 0.124 to 0.176 ft. and with $n = 0.009$, the average value of c is 121. Here, again, it was necessary to make the model as smooth as possible, although the effect of friction is a minor factor.

During the course of the tests, certain phenomena appeared, which it was desirable to interpret as to their probable physical magnitudes in the full-sized spillway. These were (*a*), the "mushroom" of violently agitated water formed in the cone-shaped space above the freely falling, impinging jet (Figs. 7 and 9); (*b*), the projecting jet of water formed opposite the "nose" of the spiral approach channel in the case of the model of the clear crest type, due to the spiral movement of the water (Figs. 9 and 19); and (*c*) the partial vacuum in the lower portion of the freely falling jet. The following approximate interpretations have been made.

(a).—Mushroom of Disturbed Water.—As all flows in the model are $\left(\frac{1}{36}\right)^{\frac{5}{2}}$ times those in the full-sized spillway and all velocities are $\left(\frac{1}{36}\right)^{\frac{1}{2}}$ times full-sized velocities, and as the power loss due to shock or impact must be

proportional to $Q(v')^2$, it follows that the power loss ratio is $\left(\frac{1}{36}\right)^{\frac{7}{2}}$. If this lost power is expended in maintaining in motion a "mushroom" of water of height, x_1, above the point where the jet closes in on itself, the volume of this mushroom varies as x_1^3, whereas the average height of throw varies as x_1. The power expended in maintaining the mushroom, therefore, varies as x_1^4, from which it is to be expected that the height ratio of the "mushroom" would be $\left(\frac{1}{36}\right)^{\frac{7}{8}}$, or about $\frac{1}{23}$; in other words, at corresponding stages of flow, the height of the mushroom in the full-sized spillway will not be thirty-six times, but about twenty-three times, the height on the model.

(b).—Projecting Jet, Clear Crest Model.—The lost power used in projecting this jet horizontally is undoubtedly proportional to $Q_2 v_2^2$, where Q_2 is the flow which reaches the spillway through the spiral approach channel and v_2 is the tangential velocity in the channel. Evidently, the power loss ratio is $\left(\frac{1}{36}\right)^{\frac{7}{2}}$, as in the case of the "mushroom", but, in this case, the power is used rather to increase by an unknown amount the horizontal velocity of an unknown volume of the flow near the nose. There seems to be no way of determining just how these two unknown factors will arrange themselves, in order to account for the lost power. If the flow ratio for this projected jet is $\frac{1}{7\,776}$, and the velocity ratio is $\frac{1}{6}$, the "power expended" ratio will be the same as the "power lost" ratio, and the dimensional ratio for the projected jets will be $\frac{1}{36}$, the same as the dimensional ratio for the model itself, but this is only one of an infinite number of possible sets of ratios.

(c).—Partial Vacuum.—As explained in the case of the first design of the spillway, the pressure within the freely falling jet section down as far as the top of the vertical cylindrical section should be atmospheric. In order to check this, at first a single piezometer was connected to the model at the top of the cylindrical shaft. When the first day's experiments at Worcester indicated that, for flows somewhat in excess of the design maximum, a considerable vacuum existed, it was decided to study further the pressure conditions within the model; therefore, four additional piezometers were installed at points shown in Fig. 2. Succeeding observations plainly indicated that, as far as the model was concerned, the stream of water, after the jet closed in, accelerated more than had been anticipated so that the jet was smaller than the shaft of the model. For some unknown reason, there was a natural Venturi tube effect, and the velocity head in excess of that assumed in the design must be regained as pressure head beyond the last piezometer, for if it was lost in shock or eddies there would not be sufficient pressure head remaining to account for the observed flow. Although all these conclusions were obvious in the case of the model, the

very fact that no reason could be discovered for this Venturi effect precludes any interpretation of the model results as far as the full-sized spillway is concerned. It is believed that there will be a partial vacuum in the full-sized spillway when the flow is equal to or approaching the maximum capacity, but the extent of the vacuum cannot be predicted. If the dimensional ratio of the Venturi jets should be the same as that between the model and the full-sized spillway, it can be shown that the vacuum ratio has the same value. However, this could only be the case for measured vacuums of 0.944 ft., or less, on the model for, obviously, a vacuum in excess of 34 ft. cannot be obtained in the full-sized spillway.

4.—Cylindrical Shaft, Bend, Transition, and Standard Tunnel Section.— If this part of the model is built to $\frac{1}{36}$ of the linear dimensions of the full-sized spillway with the sole exception of the length of the standard tunnel section, it will be shown that the flow ratio of $\frac{1}{7\ 776}$ can be maintained. It is evident that the available head in this section of the model is $\frac{1}{36}$ of the head on the full-sized spillway. Furthermore, as excess loss of head in bends and transition is assumed to vary with the velocity head, $\frac{V^2}{2g}$, it is seen that if the velocity in the bend and transition is $\frac{1}{6}$ of that in the full-sized spillway, the ratio of excess losses will be $\frac{1}{36}$, the same as the model ratio. The remaining problem, therefore, is to make the total friction loss ratio also $\frac{1}{36}$.

If the Chezy formula for friction loss is used, the total loss in this section due to friction varies as $\frac{V^2\,l}{c^2\,R}$, in which l = the total length of the section and R = the hydraulic radius. In order to obtain a total friction loss ratio of $\frac{1}{36}$, with a velocity ratio of $\frac{1}{6}$ and a hydraulic radius ratio of $\frac{1}{36}$, then $\frac{l}{c^2}$ for the model must be $\frac{1}{36}$ of $\frac{l}{c^2}$ for the full-sized spillway. The value of R for the latter is 5.3 ft. which, with $n = 0.014$ in Kutter's formula, corresponds to $c = 136.5$. The value of l for the full-sized spillway being 957 ft., it follows that $\frac{l}{c^2}$ for the model must equal $\frac{1}{36} \times \frac{957}{136.5^2}$. The hydraulic radius for the model being 0.147 ft., the values for n and l given in Table 2, can be obtained for the model.

If not distorted, l would be 26.3 ft. Inasmuch as it was desired to have the minimum amount of distortion, it was decided to make this whole section as smooth as possible (for $n = 0.009$) and to use a length of 17 ft. for the standard tunnel section. The final dimensions of the model are given on Fig. 2.

TABLE 2.

Kutter's n.	Chezy's c.	Total length of section, l, in feet.	Length of standard tunnel section, in feet.
0.009	120	20.4	17.0
0.010	105	15.6	12.2
0.011	93	12.2	8.8
0.012	83	9.8	6.4
0.013	75	8.0	4.6
0.014	68	6.55	3.15

Tests on Model

The standard tunnel section of the model was constructed of galvanized iron sheets, the remainder, including the bridge piers, being of wood and all the surfaces over which water flowed being well varnished. The level floor of the approach channel was made of unplaned boards, and the curved surface connecting the spillway lip with this floor was built up with coarse sand mortar, intentionally roughened. It has been mentioned that in the design of the spiral approach for the full-sized spillway, the value of n in Kutter's formula was taken as 0.035. When the reduction in hydraulic radius of the model is taken into account, it is found that the surfaces of the spiral approach for the model must have a roughness corresponding to $n = 0.017$, in order to maintain the desired flow ratio, and, for this purpose, the vertical walls of the approach channel were made of galvanized iron, coated with heavy paint into which coarse sand had been thrown before the paint dried. As mentioned previously, the spillway model proper was made as smooth as practicable, in order to attain a coefficient of roughness, $n = 0.009$, if possible, but it should be noted that failure to secure so low a coefficient is on the side of safety, as it would decrease the flow through the spillway model under the given heads and thus indicate conservative values of the capacity for the full-sized spillway under the corresponding heads. The general appearance of the model is shown in Fig. 16.

As the radial dimensions of the spiral approach channel for the clear crest type differed from those for the flash-board pier type, it was necessary to construct the model of the channel in such a way as to enable both channels to be tested. In order to facilitate the work of the tests, the model was constructed to permit the shift from one spiral channel to the other to be made very quickly. The flash-board piers were attached by dowels, and so could be placed or removed readily. The galvanized-iron strips used for the vertical walls of the approach channel were cut off on a slope, corresponding to the slope of the ground at the spillway site, to a point well above the crest of the model. The outstanding edges of the wooden floor of the approach channel were also sloped down for about 2 in. at an angle corresponding to the ground slope, in order to simulate the full-sized approach-channel conditions and to avoid the contraction that would be caused by an abrupt edge at this point. The position of the approach model was such that it bore the same relation to the tunnel model as that which obtains in the full-sized spillway.

As at first constructed, the pier models extended slightly beyond the theoretical down-stream edge of the converging flat crest weir section, but during the

progress of the Whitingham tests the inner ends were cut off until the length of the piers was only 4.8 in.

In all cases, the flow through the spillway model was measured by a sharp-edged weir, and the heads both on this measuring weir and on the crest of the spillway model were measured by hook-gauges located in stilling-boxes. Pressures within the spillway were measured in open-end, vertical, glass tubes attached by rubber tubing to small flexible copper tubing leading to the piezometer connections. Depths of water at the discharge end of the tunnel were determined by measuring with a rule from the crown of the tunnel to the water surface. Profiles for the water surface on the clear crest model were determined by measuring horizontally along a level rod (Figs. 7 and 8) fixed in place and then vertically downward. Some of the profiles in the case of the model of the pier crest type were taken by marking the water-line on the piers with a lead pencil. Studies to determine the best shape for the pier ends were made by observing the contraction conditions during the tests and by marking the lines of contraction on the flat surface of the spillway model between the piers.

Worcester Tests of June 7, 1922.—The purpose of the first two series of tests on the model, carried out at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, at Worcester, Mass., on June 7 and 9, 1922, was to ascertain first, whether the capacity of the spillway without a spiral approach channel agreed with the predicted capacity at the design head of 7 ft.; and, second, whether the water would be likely to enter the upper end of the shaft in such a disturbed state of flow as to produce undue losses of head in the shaft and tunnel and thus possibly cause erosion of the concrete lining. As described previously, the curve of the spillway was designed so that water passing over the crest under the specified head of 7 ft., would adhere everywhere to the face of the spillway, as it was believed that this was a condition precedent to a small amount of disturbance. As experimental data available for fixing the profile of the spillway were rather meager, this point was carefully observed during the tests.

If the model proved satisfactory in regard to these two main objects, it was planned to obtain experimental profiles of the water surface for various heads on the crest of the model ranging from 0 in. to 5 in. (the latter depth corresponding to one of 15 ft. on the full-sized spillway), and to make observations on the pressure conditions within that part of the spillway above the 90° bend. Fortunately, all these projects were successfully undertaken.

Fig. 5 shows a plan and longitudinal section of the testing flume used at Worcester, and a general view of the equipment looking up stream toward the leading flume is given in Fig. 6. It will be noted that there was a considerable drop in the floor between the two flumes, which caused the water to accelerate as it entered the testing flume. This, together with the fact that the normal velocities in the leading flume were already rather high, led to a considerable disturbance of the water surface in the testing flume. An attempt was made to eliminate this disturbance by means of a wooden screen placed across the entrance to the flume and, although this proved beneficial, it did not eliminate

the whole trouble. The disturbed condition of the water surface is clearly shown on the photographs, Figs. 7 and 8, taken during the tests, both by the irregularities or ripples and by the bubbles coming to the surface due to the entraining of air as the water passed through the stilling-rack, where it was violently agitated. Tests made on the flume indicated that it was sufficiently tight to justify neglecting leakage as a factor in the work.

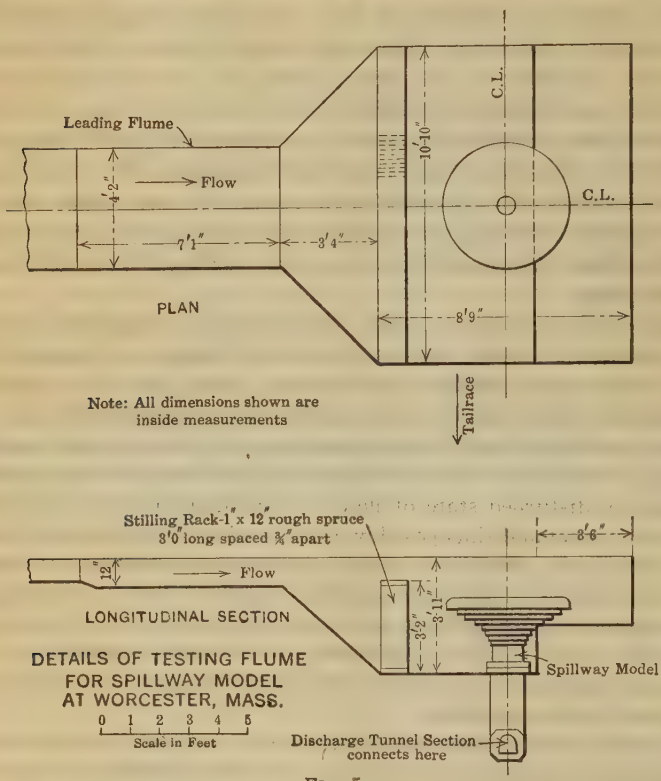

FIG. 5.

The water supplied to the leading flume was measured by a rectilinear, thin-edged notch which, together with its channel of approach, is fully described in Table 3 of experimental data for the tests of June 7, 1922. On the authority of C. M. Allen, M. Am. Soc. C. E., who has had extensive experience with this apparatus, the Hamilton Smith weir formula was used for computing this flow. Sluice-gates on either side of the measuring weir were used to by-pass water directly into the tail-race and in this way to regulate the flow into the leading flume. Due to the fact that other experiments were being conducted on the same day, the discharge from which formed the supply for the spillway tests, difficulty was encountered in keeping a constant flow through the spillway model for any length of time, and, therefore, the number of observations was limited. Photographs were taken from a platform immediately over the center of the model (Fig. 6), the camera pointing vertically downward.

FIG. 6.—VIEW OF TESTING FLUME AND SPILLWAY MODEL, WORCESTER, MASS.

FIG. 7.—SPILLWAY MODEL IN WORCESTER FLUME UNDER DESIGN HEAD OF 2.32 INCHES.

FIG. 8.—SPILLWAY MODEL IN WORCESTER FLUME UNDER HEAD OF 10.25 INCHES, SHOWING VORTEX.

FIG. 9.—SPILLWAY MODEL WITH SPIRAL APPROACH CHANNEL IN WHITINGHAM, VT., FLUME, AT DISCHARGE OF 2.35 SECOND-FEET.

The experimental data for these tests and their analyses are shown in Tables 3 and 4, and, from these data, the capacity curve shown on Fig. 10 was plotted. This curve represents the capacity of the clear crest type of model set in deep water, that is, without a spiral approach channel. These first tests showed conclusively that the capacity of the spillway with a depth of 2.33 in. on the crest was close to the predicted capacity, being about 96% of that amount. The tests also indicated that this was not the maximum, but that, apparently, its practical limit was reached with a depth of about 2.89 in. on the crest, at which time the flow was about 20% greater than the nominal maximum capacity. It was thus seen that if the same increase were maintained in the full-sized spillway the maximum capacity would be about 32 000 sec-ft. and that the corresponding depth on the spillway would be about 8.67 ft.

TABLE 3.—EXPERIMENTAL DATA ON WORCESTER TESTS OF JUNE 7, 1922.

General:

Mean elevation of crest of spillway model = 1.281 ft. on model hook-gauge.
" " " " measuring weir = 0.650 " " weir " "
Reading of 3.900 on scale of throat piezometer = reading of 2.000 on model hook-gauge.

NOTE: All scales are set with zero at lower end, that is, readings increase upward.

Measuring Weir:

This is a rectangular, thin-edged notch, that is, it has end contractions. The length of the weir is 3 ft. 6$\frac{3}{32}$ in. at the bottom of the notch, and it has the same length at a distance of 6 in. above that point. This covers the range of heads used on the weir. The cross-section of the channel of approach, at the point where the connection is made to the stilling-box, in which the weir hook-gauge is set, was measured at a time when the water in the channel was 0.2 ft. below the crest of the measuring weir with the following results:

Distance from initial point.	Depth of water.
10 ft. 6 in.	0.0 ft.
10 " 0 "	2.5 "
7 " 6 "	1.725 "
5 " 7 "	0.6 "
2 " 8 "	0.525 "
0 " 0 "	1.0 "

Time.	Model hook-gauge.	Weir hook-gauge.	Throat piezometer reading.	Remarks.
A. M.	1.450 ft.	1.044 ft.	.....	.....
	1.439 "	1.026 "	.....	
A. M.	1.418 ft.	0.969 ft.	.....	.....
	1.416 "	0.967 "	.....	
A. M.	1.351 ft.	0.810 ft.	.....	.....
P. M.	1.499 ft.	1.123 ft.	Range, 0.30 to 0.17 ft.	Tunnel nearly full.
	1.495 "	1.122 "	Average, 0.22 ft.	
P. M.	1.522 ft.	1.160 ft.	Range, 0.05 to 0.08 ft.	Tunnel full.

These first tests solved the problem as to the form of disturbance that would be caused by the loss of the horizontal velocity of the jet. At heads as low as 1.64 in. on the model, corresponding to 4.92 ft. on the full-sized spillway, the jet began to close in, forming a small disturbed "mushroom" far down in the throat of the spillway. This mushroom grew both in volume and height from that head, being very distinct from the flowing jet of water which could be seen to slide smoothly over the spillway model (see Figs. 7, 9, and 19), and under this disturbed part. At some head between 2.62 in. and 2.87 in., this distinction between the mushroom and the flowing jet began to disappear until, at the latter head, corresponding to the practical maximum capacity of the model, the spillway was entirely flooded, acting practically as a vertical orifice. As

TABLE 4.—Analysis of Experimental Data on Worcester Tests of June 7, 1922.

(Length of Measuring Weir = 3.508 Ft.)

Length of crest of model, measured along up-stream edge = 13.963 ft.; B (for model) = 1.0625 ft.

Vacuum at Throat of Flaring Section (Piezometer No. 4):

Head on model.	Q.	Vacuum, in inches of water.
0.216 ft. = 2.59 in.	3.72 sec-ft.	9.76
0.241 " = 2.89 "	4.16 "	13.18

No.	Head on measuring weir, D, in feet.	Value of C.	Value of $D^{3/2}$.	Approximate value of Q.	Cross sectional area of channel of approach, in square feet.	Approximate velocity of approach, in feet per second.	Velocity head, h_v, in feet.	Head on model.	Value of $H^{3/2}$.	Value of C for model.	Value of $\frac{H}{B}$ for model.
1	0.394	3.284	0.2474	2.85	18.3	0.156	0.0004	0.169 ft. = 2.03 in.	0.0695	2.94	0.159
2	0.376	3.290	0.2306	2.66	18.2	0.146	0.0003	0.158 " = 1.90 "	0.0628	3.04	0.149
3	0.319	3.310	0.1802	2.10	17.6	0.119	0.0002	0.137 " = 1.64 "	0.0508	2.96	0.129
4	0.317	3.311	0.1785	2.07	17.5	0.118	0.0002	0.135 " = 1.62 "	0.0496	2.99	0.127
5	0.150	3.416	0.0581	0.70	15.8	0.044	0.0000	0.070 " = 0.84 "	0.0185	2.71	0.066
6	0.473	3.264	0.3253	3.73	19.2	0.194	0.0006	0.218 " = 2.62 "	0.1018	2.63	0.205
7	0.472	3.265	0.3248	3.72	19.2	0.194	0.0006	0.214 " = 2.57 "	0.0990	2.69	0.202
8	0.510	3.256	0.3642	4.16	19.6	0.212	0.0007	0.241 " = 2.89 "	0.1183	2.52	0.226

Note.—The velocity heads of approach were found to be so small as to be negligible, hence the approximate values of Q are also the final values of Q. Values of C in the weir formula, $Q = C L H^{3/2}$ were taken from the Hamilton Smith tables (See *Water Supply Paper No. 200*, U. S. Geological Survey, p. 36.).

the experimental capacity agreed so closely with the predicted capacity, it was evident that this mushroom of disturbed water had no back-water effect tending to decrease the discharge, but simply represented a volume of water maintained in violent motion by the energy continuously being lost in impact as the jet closes in on itself, destroying the horizontal velocity. The jet of water issuing from the end of the tunnel showed a comparatively small volume of entrained air, and the check on the predicted capacity proved that any air so entrained had only a negligible effect, if any, on the capacity.

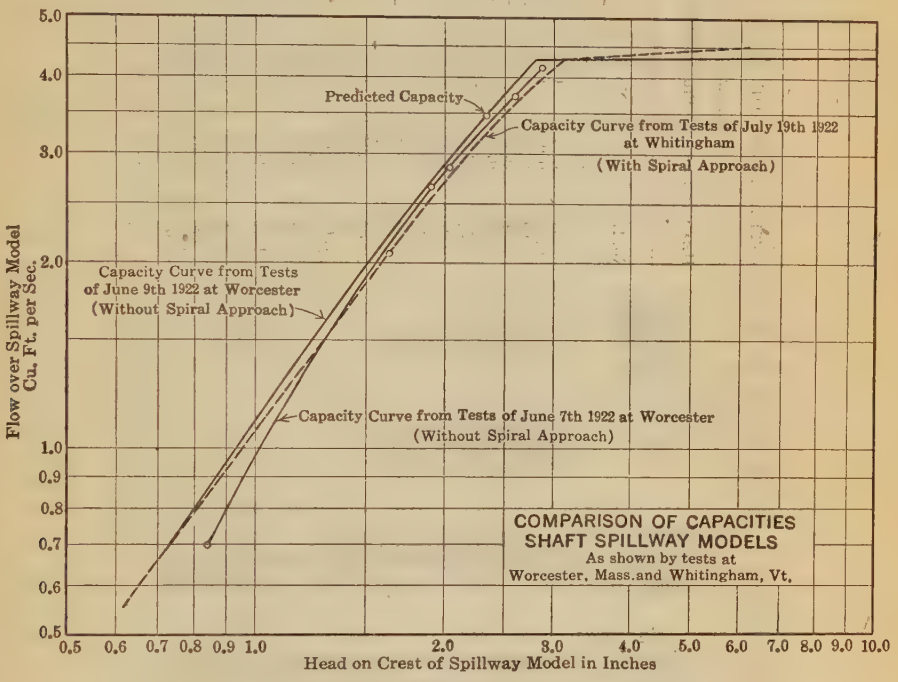
Predicted Capacity
Capacity Curve from Tests of July 19th 1922 at Whitingham (With Spiral Approach)
Capacity Curve from Tests of June 9th 1922 at Worcester (Without Spiral Approach)
Capacity Curve from Tests of June 7th 1922 at Worcester (Without Spiral Approach)
COMPARISON OF CAPACITIES SHAFT SPILLWAY MODELS
As shown by tests at Worcester, Mass. and Whitingham, Vt.
Flow over Spillway Model Cu. Ft. per Sec.
Head on Crest of Spillway Model in Inches
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.5 0.6 0.7 0.8 0.9 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

FIG. 10.

These first tests also showed clearly that the water at no time left the surface of the spillway and that, therefore, the shape of the spillway was satisfactory from that standpoint. However, the one piezometer tube which had been placed at the upper end of the cylindrical section of the spillway shaft, and on which observations were made for the last three runs on June 7, indicated a vacuum of from 9.76 to 13.18 in. of water at this point, when the spillway was operating at or near its maximum capacity. As already stated under "Design" (page 8), it was assumed that the pressure at this point, as well as that at all points above it, would be atmospheric. In view of the indications of the existence of a partial vacuum, it was decided to install four additional piezometer tubes before making further capacity tests and taking profiles of the water surface. The locations of these additional tubes, as well as the location of the original tube, are shown on Fig. 2.

TABLE 5.—Analysis of Experimental Data on Worcester Tests of June 9, 1922.

Time, P. M.	Head on measuring weir, D, in feet.	Hamilton Smith coefficient for measuring weir, C.	$D^{\frac{3}{2}}$.	Q, over measuring weir, in cubic feet per second.	Head on crest of spillway model, H, in feet.	$H^{\frac{3}{2}}$.	Computed coefficient for spillway model.	Head, in feet, at Piezometer Connection: No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	Q_1, over spillway model (when different from Q).
1:35	0.349	3.298	0.2062	2.38	0.143	0.0541	3.15	0.00	0.01	—0.03	—0.24	0.21	....
40	0.365	3.293	0.2206	2.54	0.149	0.0575	3.16	0.00	0.01	0.05	—0.24	0.20	....
45	0.377	3.289	0.2317	2.66	0.154	0.0605	3.15	0.00	0.02	0.01	—0.23	0.18	....
50	0.394	3.284	0.2474	2.84	0.165	0.0670	3.04	0.01	0.35	0.09	—0.31	0.17	....
55	0.410	3.279	0.2625	3.02	0.174	0.0726	2.98	0.05	0.44	0.09	—0.34	0.18	....
2:00	0.450	3.269	0.3019	3.46	0.194	0.0854	2.91	0.17	0.48	—0.23	—0.64	—0.45	....
05	.....	.....	.....	...	0.214	.....	....	0.24	0.47	—0.36	—0.89	—0.72	....
10	0.487	3.261	0.3398	3.88	0.212	0.0976	2.85	0.24	0.50	—0.40	—0.89	—0.67	....
15	0.488	3.261	0.3409	3.90	0.237	.....	....	0.23	0.49	—0.43	—0.88	—0.69	....
20	0.489	3.260	0.3420	3.90	0.214	0.0990	2.81	0.27	0.52	—0.41	—0.91	—0.67	....
25	0.510	3.256	0.3642	4.15	0.237	0.1082	2.74	0.42	0.57	—0.35	—1.29	—0.72	....
30	0.522	3.254	0.3772	4.30	0.253	0.1273	2.42	0.45	0.59	—0.40	—1.29	—1.02	....
33	0.533	3.252	0.3891	4.43	0.625	.....	....	0.87	0.95	—0.13	—1.14	—0.67	4.33
36	0.556	3.248	0.4146	4.71	0.771	.....	....	1.04	1.06	—0.03	—1.21	—0.42	4.34
38	0.567	3.247	0.4269	4.86	0.917	.....	....	1.15	1.10	0.05	—1.09	—0.42	4.34
42	0.584	3.244	0.4463	5.07	1.279	.....	....	1.71	1.63	0.42	—0.99	—0.67	4.35
55	0.636	3.237	0.5072	5.75	1.49	.....	....	1.72	1.70	0.32	—0.97	—0.25	4.35
3:03	0.688	3.230	0.5644	6.37	1.70	.....	....	1.92	1.85	0.49	—0.95	—0.27	4.35
05	0.646	3.235	0.5192	5.88	1.62	.....	....	1 84	1.76	0.37	—1.01	—0.25	4.35
07	0.563	3.248	0.4225	4.81	1.50	.....	....	1.67	1.65	0.23	—0.99	—0.30	4.35
08	.....	.....	.....	....	1.42	.....	....	....	....	....	....	....	4.35
10	0.537	3.252	0.3935	4.48	1.33	.....	....	1.60	1.64	0.52	—0.66	—0.37	4.35
13	0.526	3.253	0.3815	4.35	1.35	.....	....	1.59	1.64	0.52	—0.69	—0.62	4.35
16	0.496	3.259	0.3494	3.99	1.27	.....	....	1.50	1.58	0.65	—0.57	—0.22	4.35
18½	0.463	3.266	0.3151	3.61	0 833	.....	....	0.94	1.10	0.22	....	—1.02	4.34

TABLE 5.—(*Continued.*)

Time, P. M.	Head on measuring weir, D, in feet.	Hamilton Smith coefficient for measuring weir, C.	$D^{\frac{3}{2}}$.	Q, over measuring weir, in cubic feet per second.	Head on crest of spillway model, H, in feet.	$H^{\frac{3}{2}}$.	Computed coefficient for spillway model.	Head, in Feet, at Piezometer Connection:					Q_1, over spillway model (when different from Q).
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	
3:20	0.460	3.266	0.3120	3.58	0.451	.....	....	0.54	0.74	0.07	—0.54	—0.82	4.32
21½	0.453	3.268	0.3049	3.50	0.229	0.1096	...	0.42	0.60	—0.03	—0.69	—0.72	4.15
23	0.449	3.269	0.3009	3.46	0.207	0.0942	....	0.27	0.53	—0.11	—0.59	—0.62	3.72
25	0.442	3.271	0.2939	3.37	0.194	0.0854	....	0.30	0.55	—0.09	—0.54	—0.62	3.48
28	0.432	3.273	0.2840	3.26	0.189	0.0822	....	0.22	0.52	0.00	—0.44	—0.47	3.32
31	0.420	3.276	0.2722	3.12	0.181	0.0770	....	0.17	0.53	0.09	—0.32	—0.37	3.17
34	0.409	3.279	0.2616	3.01	0.175	0.0732	2.95	0.12	0.52	0.11	—0.29	—0.20	....
37	0.402	3.281	0.2549	2.94	0.173	0.0720	2.92	0.04	0.48	0.07	—0.26	—0.12	....
40	0.407	3.280	0.2596	2.98	0.175	0.0732	2.92	0.04	0.46	—0.01	—0.29	—0.17	....
43	0.222	3.359	0.1046	1.23	0.087	0.0257	3.43	0.00	0.00	—0.05	—0.14	0.13	....
46	0.271	3.330	0.1411	1.65	0.110	0.0365	3.24	0.00	0.00	—0.06	—0.13	0.13	....
49	0.235	3.350	0.1140	1.34	0.095	0.0293	3.28	0.00	—0.08	—0.05	—0.10	0.11	....
52	0.212	3.364	0.0976	1 15	0.089	0.0266	3.10	0.00	—0.08	—0.04	—0.16	0.13	....
55	0.197	3.377	0.0874	1.03	0.079	0.0222	3.32	0.00	—0.10	—0.04	—0.14	0.13	....
58	0.250	3.341	0.1250	1.47	0.104	0.0336	3.14	0.00	—0.08	—0.05	—0.11	0.16	....
4:01	0.161	3.407	0.0645	0.77	0.065	0.0166	3.32	0.00	—0.08	—0.02	—0.18	0.11	....
04	0.160	3.408	0.0640	0.76	0.064	0.0162	3.36	0.00	—0.08	—0.04	—0.13	0.10	....
07	0.162	3.407	0.0652	0.78	0.065	0.0166	3.36	0.00	—0.08	—0.04	—0.15	0.08	....
10	0.158	3.410	0.0628	0.75	0.065	0.0166	3.23	0.01	—0.08	—0.03	—0.14	0.08	....
13	0.154	3.413	0.0605	0.72	0.063	0.0158	3.26	0.00	—0.08	—0.04	—0.16	0.09	....
16	0.156	3.411	0.0616	0.74	0.063	0.0158	3.36	0.00	—0.08	—0.04	—0.15	0.08	....
19	0.160	3.408	0.0640	0.76	0.065	0.0166	3.28	0.00	—0.08	—0.04	—0.11	0.09	....
22	0.156	3.411	0.0616	0.74	0.064	0.0162	3.27	0.00	—0.08	—0.03	—0.12	0.08	....
25	0.154	3.413	0.0605	0.72	0.062	0.0155	3.32	0.00	—0.08	—0.04	—0.16	0.11	....
28	0.158	3.410	0.0628	0.75	0.064	0.0162	3.31	0.00	—0.08	—0.04	—0.16	0.11	....
30	0.155	3.412	0.0610	0.73	0.064	0.0162	3.23	0.00	—0.08	—0.03	—0.12	0.10	....

For the purpose of comparison with the discharge coefficients for flat crest weirs shown on Fig. 4, the discharge coefficients for the spillway model were computed and are given in Table 4. It is seen that for heads less than the design head of 7 ft., that is, for ratios of $\frac{H}{B}$ less than 0.183, the model coefficients are much higher than those given by the curve, this difference being due to the sloping crest of the model which, for small values of $\frac{H}{B}$, has a slope such that it much more than compensates for the convergence. For values of $\frac{H}{B}$ greater than 0.183, the model coefficient begins to drop off rapidly as the spillway "floods out".

Worcester Tests of June 9, 1922.—Before proceeding with the second and last series of tests at Worcester, the additional piezometer tubes were installed. The conditions of flow in the channel leading to the measuring weir were very uneven on June 9, 1922, as the experiments previously mentioned were still in progress above this point. About all that could be done was to let the stream discharge as it would, increasing from a small flow over the model to a point well above that at which the model floods out and then dropping again to almost zero flow. Very complete notes on flow conditions were made, and complete sets of pressure readings on all the piezometers were taken, as well as two profiles of the water surface over the model.

After the completion of this second set of tests, it was noted that the zero readings of the hook-gauge giving depths over the spillway model differed by 0.02 ft. from the zero readings taken at the beginning of this day's experiments. The cause of this difference was carefully sought, and it was decided that a change in zero probably occurred between 2:38 and 2:42 P. M., on the afternoon of June 9, when the spillway model was flooded out, and a rule was rested on the point of the hook in order to continue the readings of the water surface. The experimental data were reduced, therefore, on this basis.

The readings for these tests are not given, but Table 5, showing the analysis of the experiments, is included. The results of these data, as far as the capacity of and the pressure within the model are concerned, are plotted in the form of curves on Figs. 11 and 12. For flows over the spillway model in excess of 3 sec-ft., the data are unreliable, due to the effect of storage in the testing and leading flumes. On account of this storage, the flow over the spillway model does not correspond to the flow over the measuring weir at the same instant. However, from a careful study of the data, it was possible to plot a fairly reliable capacity curve for heads on the crest of the model of from 0.75 in. to 16 in.

The capacity curve deduced from the tests of June 9 was also plotted on Fig. 10 for the purpose of comparing it with the capacity curve deduced from the tests of June 7. It is to be noted that the two curves are in fair agreement, and that the curve based on the test of June 9 passes exactly through the point of predicted capacity. The pressure readings, reduced to heads above or below atmospheric pressure, expressed in feet of water, have been plotted on

Fig. 12, both with the flow over the model, in cubic feet per second, and with the depth of water on crest of the model, as abscissa. It will be noted that the tests of June 9 were confirmatory of those of June 7 in so far as the existence of a vacuum at Piezometer No. 4 is concerned, and that they also showed a vacuum to exist at Piezometers Nos. 3 and 5 for certain stages of flow. The

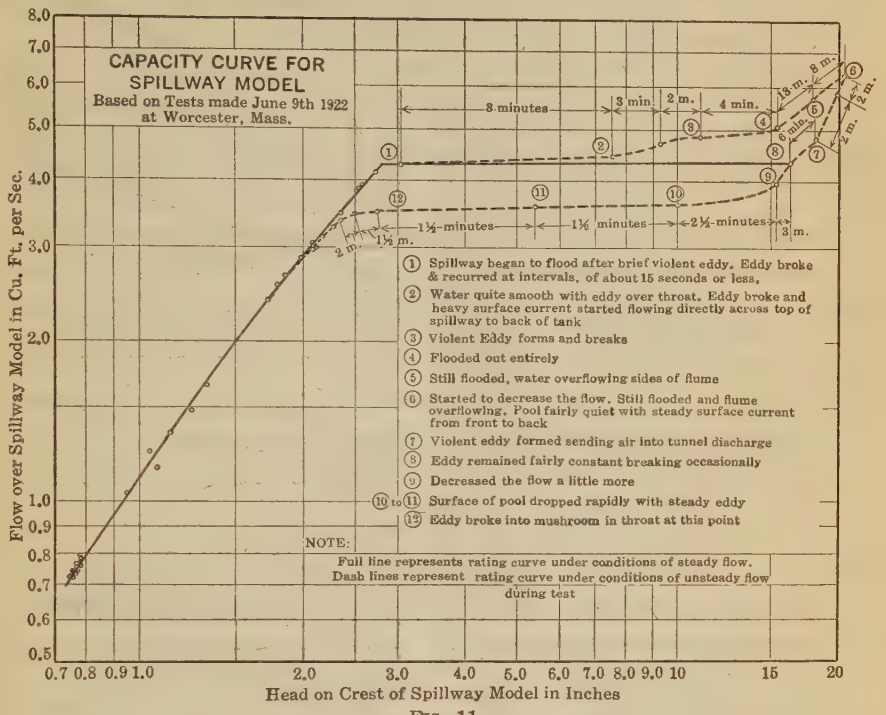
CAPACITY CURVE FOR SPILLWAY MODEL
Based on Tests made June 9th 1922 at Worcester, Mass.
Flow over Spillway Model in Cu. Ft. per Sec.
Head on Crest of Spillway Model in Inches
8 minutes
3 min.
2 m.
4 min.
6 min.
1½-minutes
1½ minutes
2½-minutes
3 m.
1½ m.
2 m.
1 Spillway began to flood after brief violent eddy. Eddy broke & recurred at intervals, of about 15 seconds or less.
2 Water quite smooth with eddy over throat. Eddy broke and heavy surface current started flowing directly across top of spillway to back of tank
3 Violent Eddy forms and breaks
4 Flooded out entirely
5 Still flooded, water overflowing sides of flume
6 Started to decrease the flow. Still flooded and flume overflowing. Pool fairly quiet with steady surface current from front to back
7 Violent eddy formed sending air into tunnel discharge
8 Eddy remained fairly constant breaking occasionally
9 Decreased the flow a little more
10 to 11 Surface of pool dropped rapidly with steady eddy
12 Eddy broke into mushroom in throat at this point
NOTE:
Full line represents rating curve under conditions of steady flow.
Dash lines represent rating curve under conditions of unsteady flow during test

FIG. 11.

curves show that the amount of vacuum is very small until a flow of from 2.5 to 3.5 sec-ft. is reached, at which time the pressure conditions seem to change rapidly. The vacuums reach maximum values at about the time the model floods out and begins to act as a vertical orifice. It will be noted from Fig. 12 that the pressure rise after the model is flooded follows, in general, a straight line, which seems reasonable for the orifice condition which then obtains.

From the pressure curves of Fig. 12, a study the details of which are given in Table 6, was made of the natural Venturi tube effect which exists in the freely falling jet and in the vertical shaft at certain stages of the flow. The computations were made for the sections of the stream of water at Piezometers Nos. 4 and 5 and indicate that, at the former section, the stream corresponding to a flow of 3.48 sec-ft. has an effective diameter of 6.94 in. and, at the latter section, 6.40 in., as compared with the shaft diameter of 7.5 in. at these two points. At the maximum flow of 4.30 sec-ft. under design conditions (that is, before "flooding", or orifice action, begins), the effective diameters of the stream at Piezometers Nos. 4 and 5 are 7.29 and 6.93 in., respectively. The discharge coefficients were computed for all the experiments of June 9, except

those obtained when the spillway was completely flooded out; the values thus obtained are shown in Table 5.

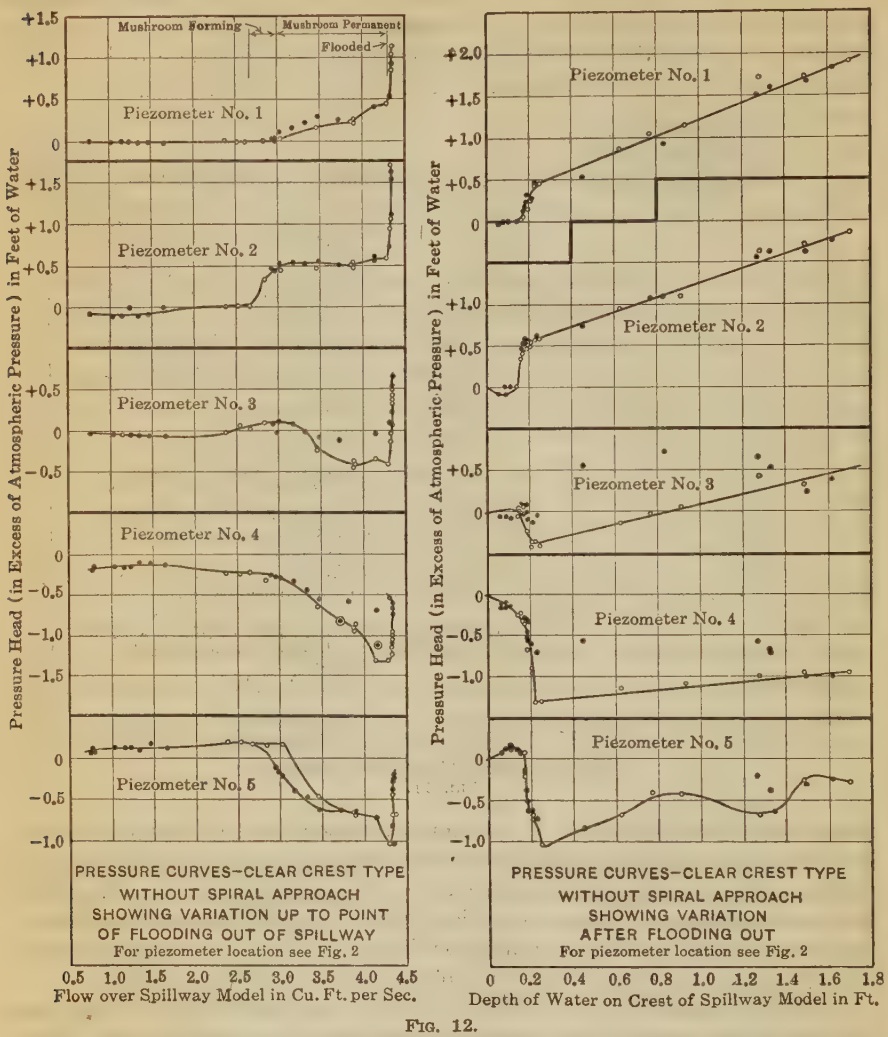
Mushroom Forming
Mushroom Permanent
Flooded
Piezometer No. 1
Piezometer No. 2
Piezometer No. 3
Piezometer No. 4
Piezometer No. 5
Pressure Head (in Excess of Atmospheric Pressure) in Feet of Water
PRESSURE CURVES—CLEAR CREST TYPE
WITHOUT SPIRAL APPROACH
SHOWING VARIATION UP TO POINT
OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
Flow over Spillway Model in Cu. Ft. per Sec.
PRESSURE CURVES—CLEAR CREST TYPE
WITHOUT SPIRAL APPROACH
SHOWING VARIATION
AFTER FLOODING OUT
For piezometer location see Fig. 2
Depth of Water on Crest of Spillway Model in Ft.

FIG. 12.

The photographs taken on June 9, 1922, as regards heads up to 2.72 in. on the crest of the model, are merely confirmatory of conditions as developed on June 7. The photographs as well as the discharge data for greater heads are merely matters of curious information, as a head of 4.33 in. on the model would correspond to a water surface at the crest of the earth dam. It is of interest to note that a vortex forms at a head of 9.25 in. on the model and is maintained in violent motion up to the highest head reached, namely, 16.42 in., which would correspond to nearly 50 ft. on the full-sized spillway. The capacity of the model, however, increases very slowly after the model floods out and

begins to act as an orifice. This is to be expected, as the discharge then varies approximately as the square root of the total difference in elevation between head-water and tail-water, and any rise in head-water creates a small increase in the total head available.

TABLE 6.—Study of Pressure and Velocity Conditions in Freely Falling Jet and in Vertical Shaft, as Developed by Worcester Tests of June 9, 1922.

For $Q = 3.48$ sec-ft. over the model, the loss of head at the edge of the spillway equals one-thirty-sixth of 0.30 ft., or 0.10 in.
For $Q = 3.48$ sec-ft. over the model, the horizontal velocity head of the free jet equals one-thirty-sixth of 8.07 ft., or 2.69 in. (A velocity head of 8.07 ft. corresponds to a velocity of 22.75 ft. per sec.)
For $Q = 3.48$ sec-ft. over the model, the head on the spillway crest equals one-thirty-sixth of 7 ft., or 2.33 in.

Gross head at Piezometer No. 4 = 2.33 + 25.77 = 28.10 in. = 2.34 ft.
" " " " No. 5 = 2.33 + 25.77 + 12.33 = 40.43 " = 3.37 "
Net head at Piezometer No. 4 = 28.10 — 2.79 = 25.31 in. = 2.11 ft.
" " " " No. 5 = 40.43 — 2.79 = 37.64 " = 3.14 "

From the curves, for $Q = 3.48$ sec-ft. over the model, there is a vacuum of about 0.6 ft. at Piezometers Nos. 4 and 5. Therefore

Velocity head at Piezometer No. 4 = 2.11 + 0.6 = 2.71 ft. Velocity = 13.24 ft. per sec.
" " " " No. 5 = 3.14 + 0.6 = 3.74 " " = 15.53 " " "

Area of jet at Piezometer No. 4 = $\frac{3.48}{13.24}$ = 0.2627 sq. ft. Diameter = 0.5783 ft. = 6.94 in.
" " " " " No. 5 = $\frac{3.48}{15.53}$ = 0.2240 " " = 0.5340 " = 6.40 "

For $Q = 4.30$ sec-ft. over the model, the head on the spillway crest equals 2.8 in., or 0.233 ft.
" " = " " " " " " " loss of head at the edge of the spillway equals 0.115 in., or 0.010 ft.
" " = " " " " " " " horizontal velocity head of the free jet assumed as 3.23 in., or 0.269 ft.

Gross head at Piezometer No. 4 = 2.80 + 25.77 = 28.57 in. = 2.38 ft.
" " " " No. 5 = 2.80 + 25.77 + 12.33 = 40.90 in. = 3.41 ft.
Net head at Piezometer No. 4 = 28.57 — 3.34 = 25.23 in. = 2.10 ft.
" " " " No. 5 = 40.90 — 3.34 = 37.56 in. = 3.13 ft.

From the curves, for $Q = 4.30$ sec-ft. over the model, there is a vacuum of about 1.3 ft. at Piezometer No. 4 and 1.05 ft. at Piezometer No. 5. Therefore.

Velocity head at Piezometer No. 4 = 2.10 + 1.3 = 3.40 ft. Velocity = 14.82 ft. per sec.
" " " " No. 5 = 3.13 + 1.05 = 4.18 " " = 16.43 " "

Area of jet at Piezometer No. 4 = $\frac{4.30}{14.82}$ = 0.2900 sq. ft. Diameter = 0.6076 ft. = 7.29 in.
" " " " " No. 5 = $\frac{4.30}{16.43}$ = 0.2616 " " = 0.5772 " = 6.93 "

It was difficult to maintain a constant flow long enough to take profiles and, for this reason, only two of any value were obtained. One of these was for a head fairly close to the design head of 7 ft. on the full-sized spillway, and the profile thus obtained has been plotted on Fig. 13, together with the predicted profile for that head. The agreement between the two profiles is fair. The greatest discrepancies occur near the lip of the spillway crest and near the upper end of the free jet. A study of the Cornell experiments on which the predicted profile was based, shows that the particular profile selected differed considerably from the ones just above and just below and this probably accounts for the first discrepancy; the other divergence is simply a matter of misjudgment in joining the upper nappe of the free jet to the computed water surface for the circular converging weir section. The highest and lowest elevations of the top of the mushroom of disturbed water are shown on this profile. The lower observed profile is also shown on Fig. 13, as a matter of interest, although no profile was predicted for the flow corresponding to it. Typical conditions of flow in the Worcester tests are shown in Fig. 7.

Whitingham Tests.—The purpose of the second two series of tests on the model, carried out in the testing flume of the Power Construction Company,

on Sadawga Brook, near Whitingham, Vt., on July 19, 20, and 21, 1922, was to ascertain, first, the effect of the spiral approach on the spillway capacity and on the flow conditions; second, the effect of flash-board piers on the spillway capacity and flow conditions; and, third, the proper length of the flash-board piers and the best shapes for the ends of these piers.

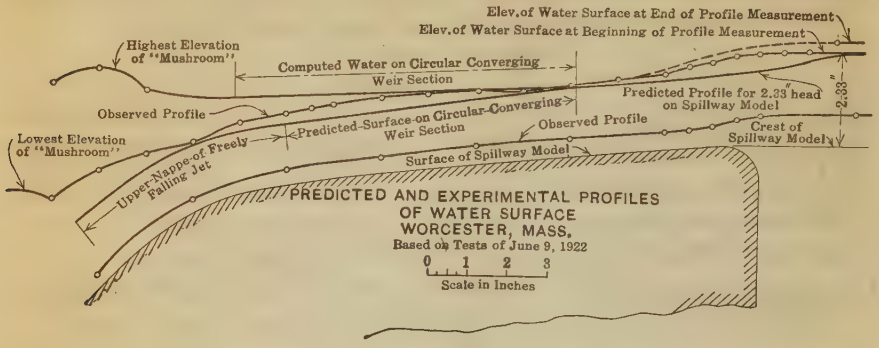
Elev. of Water Surface at End of Profile Measurement
Elev. of Water Surface at Beginning of Profile Measurement
Highest Elevation of "Mushroom"
Computed Water on Circular Converging Weir Section
Observed Profile
Predicted Profile for 2.33" head on Spillway Model
2.33"
Lowest Elevation of "Mushroom"
Predicted Surface on Circular Converging Weir Section
Observed Profile
Crest of Spillway Model
Upper Nappe of Freely Falling Jet
Surface of Spillway Model
PREDICTED AND EXPERIMENTAL PROFILES OF WATER SURFACE WORCESTER, MASS.
Based on Tests of June 9, 1922
0 1 2 3
Scale in Inches

FIG. 13.

An old crib dam on Sadawga Brook had been rebuilt to divert water into a leading flume which connects with the testing flume, shown in detail on Figs. 14, 15, and 16. Benefiting by the experience obtained at Worcester, the approach to the testing flume had been carefully designed with screens, partitions, and movable baffles, so that the water entered the flume uniformly over

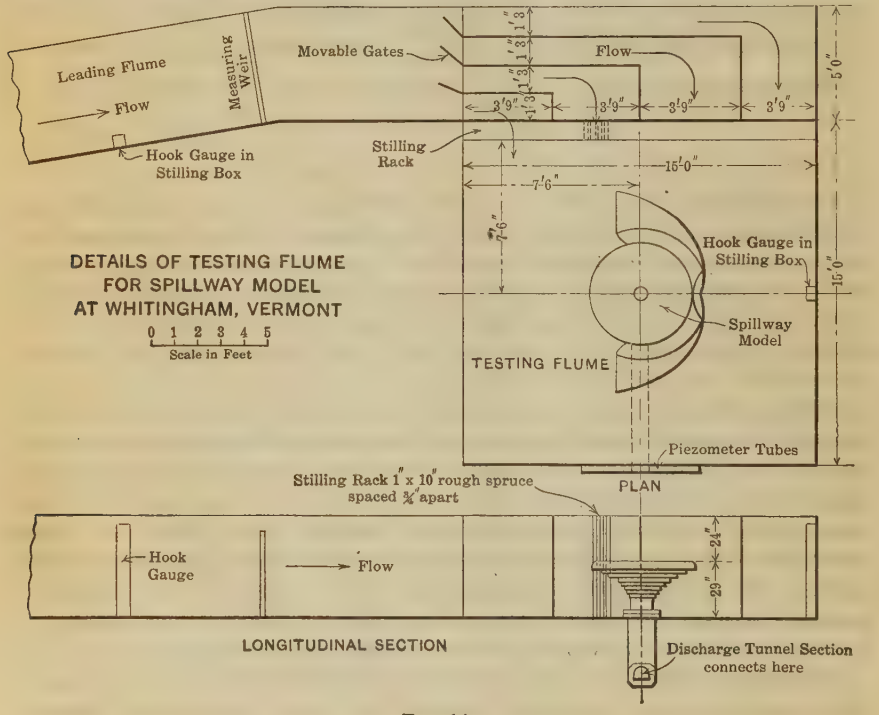

FIG. 14.

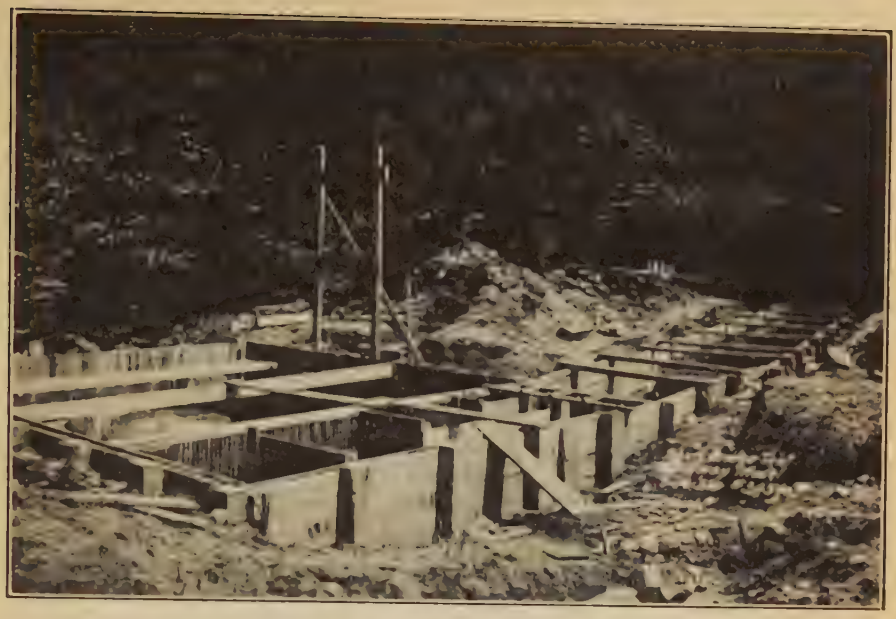

FIG. 15.—VIEW OF TESTING AND LEADING FLUMES, WHITINGHAM, VT.

FIG. 16.—VIEW OF TESTING FLUME AND MODEL, WHITINGHAM, VT.

its entire cross-section and without any disturbance whatever. Leakage tests on the flume indicated that it was very tight and justified the neglect of this factor in the tests.

The water supplied to the testing flume was measured by a thin-edged, suppressed weir, located at the lower end of the leading flume. In size, this measuring weir was similar to the Fteley and Stearns weir of 1879 and to one of the Bazin weirs. The Fteley and Stearns ratings were adopted in reducing the experimental data for these tests.

The natural flow of Sadawga Brook on the days of the experiments was just about equal to the maximum capacity of the model and was quite uniform, so that it was possible to obtain a constant flow for any length of time desired. The flow into the leading flume was regulated by sluice-gates in the dam and a head-gate at the upper end of the flume.

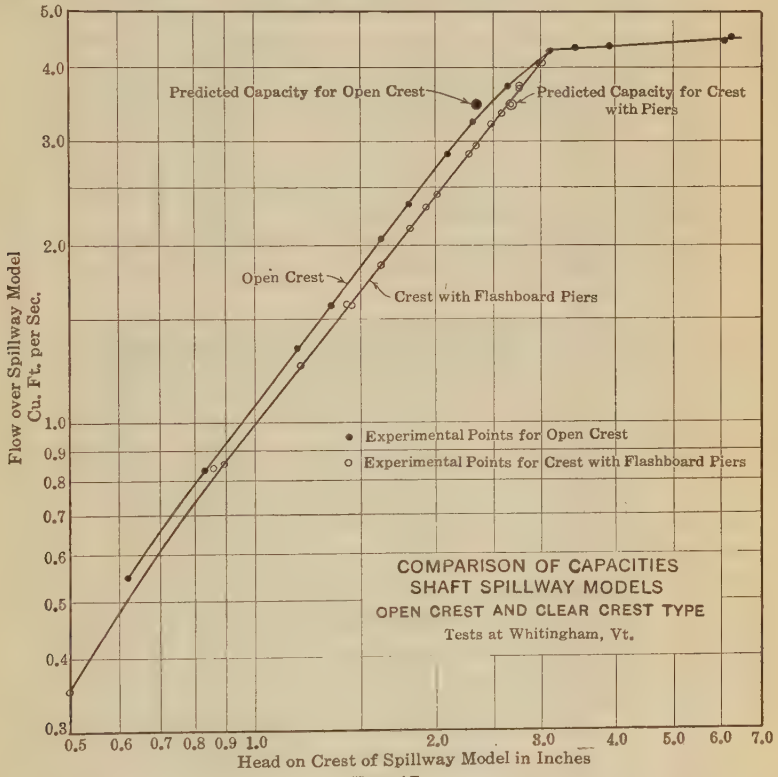

FIG. 17.

A very complete series of photographs was taken from a platform immediately over the center of the model, the camera pointing vertically downward, of which Figs. 9, 19, 21 and 22 are typical. The supports for this platform are shown on Fig. 16. In order to obtain better illumination, sunlight was reflected into the throat by two large mirrors.

The experimental data for these tests are not given, but the analyses of the experiments are given in Tables 7 and 8. The results of these tests, as far as

TABLE 7.—Analysis of Experimental Data on Whitingham Tests of July 19, 1922, on Clear Crest Model.

No.	Head on measuring weir, D, in feet.	Head on crest of spillway model, in feet.	Q, over measuring weir, in cubic feet per second.	Depth of water surface below top of tunnel model, in feet.	Area of waterway, in square feet.	Velocity at end of tunnel, in feet per second.	Depth of water in tunnel, depth of tunnel = 0.60 ft., in feet.	Head, in Feet, at Piezometer Connection:				
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
1	0.099	0.051	0.55	0.48	0.068	8.08	0.12	+0.01	+0.02	−0.02	−0.01	+0.02
2	0.131	0.069	0.83	0.43	0.100	8.30	0.17	+0.01	+0.02	−0.03	−0.02	+0.10
3	0.182	0.097	1.33	0.36	0.144	9.23	0.24	+0.00	+0.02	−0.05	−0.02	+0.18
4	0.204	0.111	1.58	0.315	0.172	9.18	0.285	+0.00	+0.02	−0.06	−0.01	+0.19
5	0.244	0.134	2.05	0.27	0.198	10.36	0.33	+0.00	+0.02	−0.04	+0.02	+0.14
6	0.267	0.149	2.34	0.25	0.210	11.15	0.35	+0.00	+0.03	+0.14(?)	−0.02	+0.14
7	0.305	0.174	2.86	0.185	0.244	11.72	0.415	+0.01	+0.21	−0.10	−0.26	−0.02
8	0.332	0.192	3.23	0.14	0.265	12.18	0.46	+0.04	+0.35	−0.20	−0.45	−0.23
9	0.365	0.219	3.71	0.025	0.302	12.28	0.575	+0.26	+0.51	−0.34	−0.67	−0.36
10	0.388	0.241	4.09	0.00	0.306	13.35	0.60	+0.40	+0.59	−0.30	−0.76	−0.36
11	0.405	0.287	4.35	0.00	0.306	14.22	0.60	+0.50	+0.63	−0.38	−0.82	−0.26
12	0.415	0.519	4.50	0.00	0.306	14.70	0.60	+0.76	+0.86	−0.24	−0.76	−0.21
13	0.411	0.509	4.45	0.00	0.306	14.54	0.60	+0.76	+0.84	−0.25	−0.77	−0.22

TABLE 8.—Analysis of Experimental Data on Whitingham Tests of July 21, 1922, on Flash-Board Pier Type of Crest Model.

No.	Head on measuring weir, D, in feet.	Head on crest of spillway model, in feet.	Q, over measuring weir, in cubic feet per second.	Depth of water surface below top of tunnel model, in feet.	Area of waterway, in square feet.	Velocity at end of tunnel, in feet per second.	Depth of water in tunnel, depth of tunnel = 0.60 ft., in feet.	Head, in Feet, at Piezometer Connection:				
								No. 1.	No. 2.	No. 3.	No. 4.	No. 5.*
1	0.174	0.099	1.25	0.38	0.132	9.47	0.22	+0.00	+0.02	−0.01	−0.02	−0.24
2	0.073	0.041	0.35	0.48	0.068	5.15	0.12	+0.01	+0.02	−0.00	−0.01	−0.23
3	0.133	0.074	0.85	0.425	0.103	8.25	0.175	+0.01	+0.02	−0.01	−0.02	−0.23
4	0.205	0.120	1.59	0.325	0.166	9.57	0.275	+0.00	+0.02	−0.02	−0.02	−0.23
5	0.251	0.150	2.14	0.285	0.190	11.26	0.315	0.00	+0.02	−0.02	−0.02	−0.23
6	0.265	0.160	2.32	0.255	0.206	11.26	0.345	0.00	+0.02	−0.04	−0.02	−0.23
7	0.306	0.188	2.86	0.195	0.239	11.96	0.405	0.00	+0.02	−0.08	−0.07	−0.23
8	0.330	0.205	3.21	0.15	0.260	12.35	0.45	−0.00	+0.01	−0.10	−0.14	−0.23
9	0.361	0.228	3.67	0.095	0.282	13.00	0.505	−0.02	+0.01	−0.22	−0.38	−0.23
10	0.388	0.248	4.09	0.02	0.303	13.48	0.58	−0.01	+0.24	−0.40	−0.68	−0.23
11	0.399	0.257	4.24	0.00	0.306	13.85	0.60	+0.42	+0.56	−0.40	−0.77	−0.23
12	0.388	0.250	4.09	0.02	0.303	13.48	0.58	−0.01	+0.18	−0.40	−0.96(?)	−0.86
13	0.406	0.325	4.35	0.00	0.306	14.22	0.60	+0.56	+0.67	−0.33	−0.74	−0.20
14	0.386	0.247	4.05	0.03	0.301	13.45	0.57	−0.00	+0.21	−0.40	−0.66	−0.94
15	0.365	0.229	3.71	0.095	0.282	13.16	0.505	−0.00	+0.02	−0.22	−0.40	−0.87
16	0.341	0.213	3.36	0.11	0.277	12.13	0.49	−0.00	+0.01	−0.12	−0.18	−0.86
17	0.312	0.193	2.94	0.20	0.236	12.45	0.40	0.00	+0.02	−0.06	−0.06	−0.86
18	0.277	0.166	2.46	0.235	0.218	11.27	0.365	+0.00	+0.02	−0.04	−0.02	−0.85
19	0.228	0.134	1.85	0.305	0.178	10.38	0.295	+0.01	+0.02	−0.02	−0.02	−0.85
20	0.205	0.118	1.59	0.335	0.159	10.00	0.265	+0.02	+0.02	−0.02	−0.01	−0.84
21	0.174	0.099	1.25	0.38	0.132	9.47	0.22	+0.01	+0.02	−0.02	−0.01	−0.84
22	0.132	0.071	0.84	0.43	0.100	8.40	0.17	+0.01	+0.02	−0.01	−0.02	−0.84

* Readings on this piezometer are unreliable.

capacity of and pressure within the model are concerned, are plotted in the form of curves on Figs. 17 and 18. The capacity curve deduced from the experiments of July 19 for the clear crest model with a spiral approach has also been plotted on Fig. 10 for the purpose of comparing it with the capacity

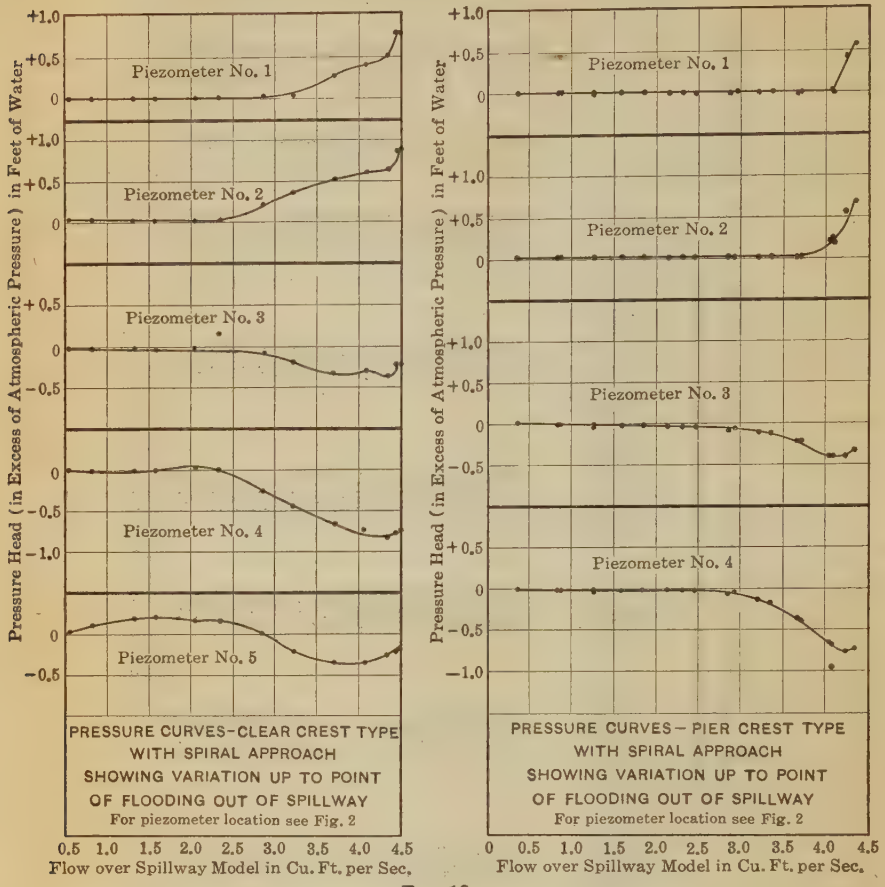
Pressure Head (in Excess of Atmospheric Pressure) in Feet of Water
Piezometer No. 1
Piezometer No. 2
Piezometer No. 3
Piezometer No. 4
Piezometer No. 5
PRESSURE CURVES—CLEAR CREST TYPE WITH SPIRAL APPROACH SHOWING VARIATION UP TO POINT OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
Flow over Spillway Model in Cu. Ft. per Sec.
PRESSURE CURVES—PIER CREST TYPE WITH SPIRAL APPROACH SHOWING VARIATION UP TO POINT OF FLOODING OUT OF SPILLWAY
For piezometer location see Fig. 2
Flow over Spillway Model in Cu. Ft. per Sec.

FIG. 18.

curves deduced from the Worcester tests on June 7 and 9, 1922. It is to be noted that the capacity curves for the tests of June 9 and July 19 are in very good agreement for flows up to 0.8 cu. ft. per sec., but that for greater flows the July 19 curve is the lower; for flows greater than 1.5 cu. ft. per sec., the July 19 curve also falls below the June 7 curve. This reduction in capacity below that obtained at Worcester is due undoubtedly to losses of head in the spiral approach and at the lip of the spillway, caused by the tangential velocity of the water. This belief is amply confirmed by the photographs taken in conjunction with the capacity tests for the clear crest model. (Figs. 9 and 19.) For flows over the model in excess of 1.34 cu. ft. per sec., a distinct projecting jet of water forms opposite the nose of the spiral approach channel, which is unquestionably

FIG. 19.—STILLWAY MODEL WITH SPIRAL APPROACH CHANNEL AT DISCHARGE OF 4.09 SECOND-FEET, SHOWING RIDGE

FIG. 20.—TYPICAL STREAM-LINE PHOTOGRAPH WITHOUT PIERS.

FIG. 21.—SPILLWAY MODEL WITH SPIRAL APPROACH AND PIERS AT DISCHARGE OF 2.42 SECOND-FEET.

FIG. 22.—SPILLWAY MODEL WITH SPIRAL APPROACH AND PIERS AT DISCHARGE OF 4.10 SECOND-FEET, SHOWING EFFECT OF PIERS ON IMPROVING FLOW CONDITIONS.

FIG. 23.—FLOW WITH FLASH-BOARDS IN PLACE, EXCEPT AT ONE SECTION BETWEEN PIERS.

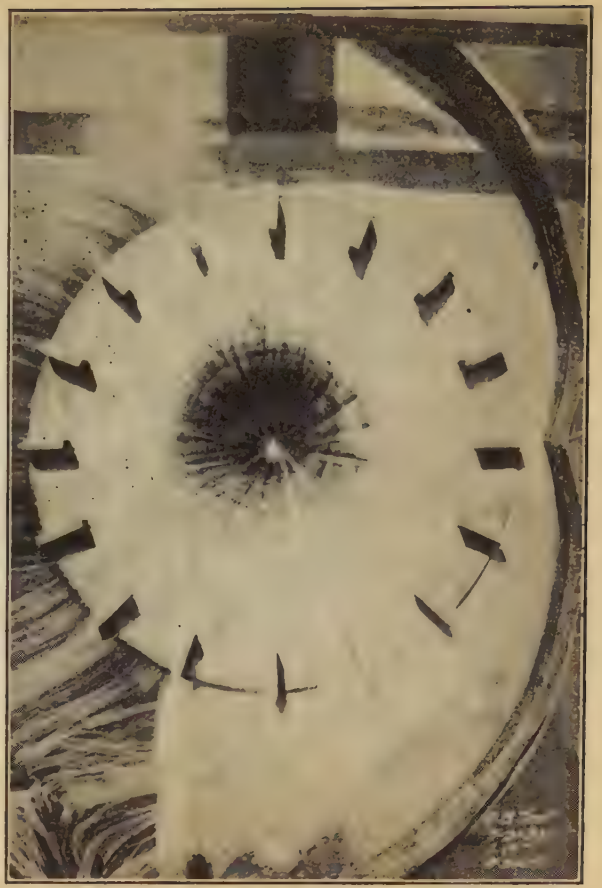

FIG. 24.—TYPICAL STREAM LINE PHOTOGRAPH WITH PIERS.

due to the tangential velocity of the water in that channel. This projecting jet, shown on Fig. 9, undoubtedly represents a considerable loss of energy and accounts in large measure for the decrease in capacity previously mentioned. On many of the photographs (see Fig. 19), the spiral motion of the water is indicated by the natural stream lines, and this is still more clearly shown on those taken during the stream-line tests (see Figs. 20 and 24), which were made by throwing shavings and sawdust into the flume and taking time exposures.

The Whitingham tests showed that the spillway floods out, that is, begins to act as an orifice, at practically the same flow, namely, 4.3 cu. ft. per sec., whether the model is provided with a spiral approach or set in open water, and whether it is provided with the flash-board piers or left with an open crest. This is as it should be, as the point at which flooding out begins is determined by the tunnel and shaft and not by conditions at the top of the spillway. The head at which the spillway floods out, however, should be determined by crest conditions, and the experiments show this to be the case.

The experimental capacity curve for the flash-board pier type of crest (Fig. 17) passes almost exactly through the point of predicted capacity for a head of 8 ft. on the full-sized spillway. This indicates that the design assumptions as to the loss of head in the spiral approach channel, at the lip of the spillway, and at the noses of the piers, must be very close to the truth. On comparing the photographs of the open-crest model with those of the flash-board type crest at the same discharges (see Figs. 9 and 21, and Figs. 19 and 22), it is noticeable that the piers have a beneficial effect in producing more uniform flow conditions around the entire spillway crest. The projecting nose of water which occurred on the open crest is nowhere in evidence. Such a projecting ridge of water (Figs. 9 and 19) would be undesirable in the full-sized spillway, especially as it is not possible to predict just how high it would rise or the amount of disturbance that would accompany it. It was decided, therefore, that the piers would be desirable even if they were not to be used to support a bridge from which to operate flash-boards. The tests having proven conclusive on this point, it was decided to adopt the flash-board pier type of crest.

At the time the Whitingham tests were made, it was proposed to use a traveling caisson or gate which would operate on a circular track around the spillway lip and serve to cut off the flow from a section of the spillway between two adjacent piers, in order to replace flash-boards on the spillway before the water had dropped below the crest. It was desirable, therefore, to determine the minimum length of pier consistent with a small quantity of back-water in a section where the flash-boards were being replaced while water was still flowing through the adjacent sections. This quantity of back-water was determined by trial, the pier models being cut off gradually until the length was only 4.8 in. With this length, there was a very small quantity of back-water at the line of flash-boards when the head on the spillway was as much as 2 or 3 in., corresponding to 6 or 9 ft. on the full-sized spillway. After the length of the piers had been determined, studies for the best shapes of these up-stream and down-stream ends were made by observing the contraction conditions at the piers and by marking the lines of contraction on the flat surface of the spillway model. For the down-stream ends of the piers, square ends, symmetrically

pointed ends, and unsymmetrically pointed ends were tried, and it was found that square ends gave the best results. For the up-stream ends both symmetrically and unsymmetrically pointed shapes were tried, and the former proved to be superior. After all these details for the piers had been settled by observation, the capacity tests for the completed pier crest type were made giving the results which are shown on Fig. 17 and from which the capacity curve was made.

In order to determine the lateral pressures on a pier due to flow on one side only, a number of profiles of water surface were taken by marking the water-line on the pier with a lead pencil. These profiles were used later in the structural design of the piers.

The question arose as to whether or not a considerable depth of flow through only one or two openings between piers would produce conditions that might cause erosion of the concrete lining. To settle this, a test, as shown in Fig. 23, was made, indicating that the flow through even one opening tends to distribute itself quite uniformly around the entire cylindrical section of the shaft and, therefore, would not be injurious.

Complete sets of pressure readings were obtained, except in the case of Piezometer No. 5, for the flash-board pier type of crest. A comparison of the resulting curves (Fig. 18), with the pressure curves, based on the Worcester tests of June 9 (Fig. 12), shows that the vacuum existing at various points in the model was, in general, less than that indicated by the Worcester tests and that of the two types of spillway the flash-board type is productive of smaller vacuums. Due to the fact that flow conditions in the Whitingham testing flume were so much better than those obtaining previously, the pressure readings also should be much more reliable.

TABLE 9.—Study of Pressure and Velocity Conditions in Freely Falling Jet and in Vertical Shaft as Developed by Whitingham Tests of July 19, 1922, on Clear Crest Model.

From the curves, for $Q = 3.48$ sec-ft. over the model, there is a vacuum of about 0.6 ft. at Piezometer No. 4 and 0.3 ft. at Piezometer No. 5. Therefore:			
Velocity head at Piezometer No. 4	$= 2.11 + 0.6 = 2.71$ ft.	Velocity	$= 13.24$ ft. per sec.
" " " " No. 5	$= 3.14 + 0.3 = 3.44$ ft.	"	$= 14.85$ ft. per sec.
Area of jet at Piezometer No. 4	$= \frac{3.48}{13.24} = 0.2627$ sq. ft.	Diameter	$= 0.5788$ ft. $= 6.94$ in.
" " " " " No. 5	$= \frac{3.48}{14.85} = 0.2342$ sq. ft.	"	$= 0.5455$ ft. $= 6.55$ in.
From the curves, for $Q = 4.30$ sec-ft. over the model, there is a vacuum of about 0.8 ft. at Piezometer No. 4 and 0.3 ft. at Piezometer No. 5. Therefore:			
Velocity head at Piezometer No. 4	$= 2.10 + 0.8 = 2.90$ ft.	Velocity	$= 13.65$ ft. per sec.
" " " " No. 5	$= 3.13 + 0.3 = 3.43$ ft.	"	$= 14.84$ ft. per sec.
Area of jet at Piezometer No. 4	$= \frac{4.30}{13.65} = 0.3150$ sq. ft.	Diameter	$= 0.6320$ ft. $= 7.58$ in.
" " " " " No. 5	$= \frac{4.30}{14.84} = 0.2896$ sq. ft.	"	$= 0.6065$ ft. $= 7.28$ in.

From the pressure curves of Fig. 18, for the clear crest model with a spiral approach channel, the effective diameters of the stream at Piezometers Nos. 4 and 5 were computed, the details of the computations being shown in Table 9. For the design flow of 3.48 sec-ft., the diameters were 6.94 and 6.55 in.,

respectively, and for the maximum flow before flooding, of 4.30 sec-ft., the diameters were 7.58 and 7.28 in., respectively. It can be stated, therefore, that, in general, the spiral approach tends to reduce the partial vacuums in the spillway and also to reduce the magnitude of the Venturi tube effect.

From the data of Tables 7 and 8 on the depth of flow at the discharge end of the tunnel model, the areas of the stream at that point were computed and the corresponding velocities obtained by dividing the areas into the observed flows. The flash-board pier type of crest having been adopted, with piers of the length and shape described, a capacity curve for the full-sized spillway was prepared from the proper curve on Fig. 17 and is shown on Fig. 25.

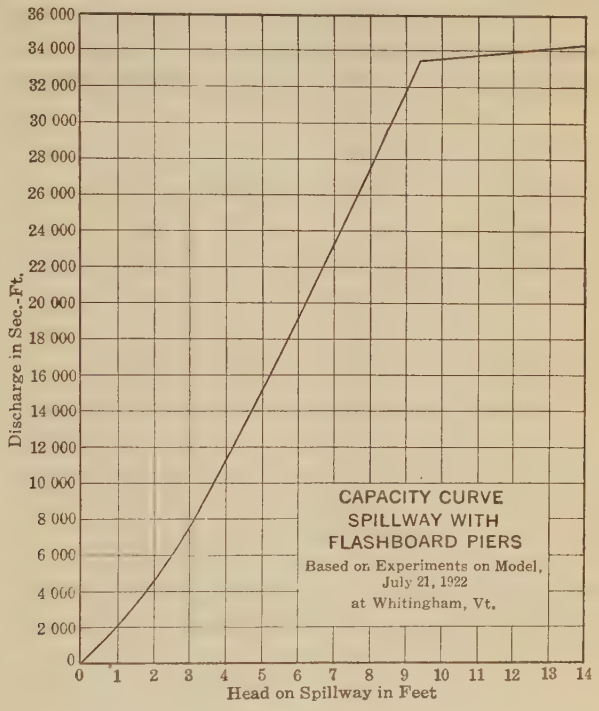

FIG. 25.

CONCLUSIONS

The writer believes that the foregoing theoretical and experimental investigations have shown that:

a.—Shaft spillways, if designed in accordance with correct hydraulic principles, can be relied on to act as predicted within the usual limits of precision of hydraulic designs involving the flow of water.

b.—The problem of the reduction in the capacity of shaft spillways due to the entraining of air is not a serious one if proper methods of design are followed.

c.—Conditions of partial or complete vacuum may exist in the converging section of shaft spillways, but they do not materially affect the capacity.

d.—If all or a section of the approach to such a spillway is to be through a comparatively narrow and shallow channel, flow conditions over the spillway will be greatly improved by the introduction of radial piers on the crest of the spillway.

Acknowledgment

The work of design and testing was done by the writer for The J. G. White Engineering Corporation, with the assistance of W. P. Creager, M. Am. Soc. C. E., and under the general direction of A. S. Crane, M. Am. Soc. C. E. The field work of testing was carried out by A. C. Eaton, M. Am. Soc. C. E., for the Power Construction Company under the general direction of the writer.

DISCUSSION

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—The author's mathematical treatment of the problem of flow into a funnel is open to question. He starts with an investigation of the flow over a broad-crested, level, flat-topped weir, and applies the resulting formulas to the converging flow over the edge of a funnel. The writer questions whether such a proceeding is warranted; assuming, however, that it is, the author's treatment of the flat-crested weir may be questioned.

Making the usual assumption that the kinetic energy of a flowing stream is some function of $\frac{V_1^2}{2g}$, where V_1 is the mean forward velocity of the stream, in feet per second, or, in other words, $V = \frac{Q}{A}$, in which A is the area of cross-section, in square feet, the principle of least energy requires that in any open channel the total energy is a minimum when $\frac{A^3}{T} = \frac{Q^2}{g}$, in which T is the width of the stream at the surface, in feet. Substituting $A\,V$ for Q, this reduces to $\frac{A}{2\,T} = \frac{V^2}{2\,g}$. If an elementary area, 1 ft. in width and Δ in depth, is considered, then $\frac{\Delta}{2} = \frac{V^2}{2\,g}$, and the elevation of the energy gradient above the bottom of the channel is $1.5\,\Delta$. This condition is now generally known as flow at the critical depth.

When water flows over a broad-crested, level, flat-topped weir, it passes over the up-stream edge at a depth depending on conditions at that point. It then flows across the weir, diminishing in depth and losing energy in the process, and flows off at the critical depth. Flow at the critical depth can exist only at the lower edge; because any other condition would involve a gain in total energy between that point and the lower edge, which is impossible. The author has arrived (page 10) at the same result in finding expressions for h and u. Writing P for $2\,\pi\,r$ in his formulas, they reduce to:

$$h = \sqrt[3]{\frac{Q^2}{P^2 g}}, \text{ and } u = \frac{Q}{P\,h} = \frac{Q}{A}$$

whence,

$$\frac{h^3\,P^3}{P} = \frac{Q^2}{g}, \text{ or } \frac{A^3}{T} = \frac{Q^2}{g}.$$

On page 11, the author gives:

$$h_1 = \frac{2}{3}\,(k\,H)$$

That is, h_1 is the critical depth for the given quantity, which can exist only at the down-stream edge. Therefore, there can be no surface drop to be accounted for and, theoretically, z must be zero. If this is true, the whole

* Cons. Engr., Vancouver, B. C., Canada.

investigation into the quantities, a and b, and the resulting equation for the parabola is based on a misconception.

However, this theoretical treatment of the critical depth is itself based on an assumption which may be very far from the truth. The kinetic energy of a flowing stream is never a simple function of V, that is, it cannot properly be represented by the velocity head calculated from the mean forward velocity, and it is this fact which accounts for the difference between the various values of C in weir formulas and the value, 3.087, obtained by the theoretical treatment. It also accounts for most of the discrepancies between prophecy and performance.

The writer also takes exception to the author's statement that "the hydraulics of the cylindrical shaft and tunnel are, of course, very simple." He questions very much whether the extension of the Kutter formula, or any other empirical discharge formula, to velocities in the magnitude of 68 ft. per sec., is warranted by the present knowledge of the facts. The writer knows of no experiments at anything like such velocities, and he doubts whether there are any, more especially for quantities as great as 27 000 sec-ft. The calculations of the head lost in curvature and transition are, also, in the writer's opinion, based on unstable foundations. Such experiments as have been recorded were made with small pipes and comparatively low velocities. It may be justifiable to extend experimental deductions over a short range, but diameters and velocities of the order treated by the author are excessive departures from the base.

In the matter of entrained air, the writer is of the opinion that a great deal more trouble will be experienced than the author apparently anticipates. The writer has seen in action an hydraulic air compressor based on the Taylor principle, in which funnel flow was the only means provided for entraining the air. It worked.

H. J. F. GOURLEY,* ESQ. (by letter).—The type of spillway, on a model of which the author has made such an interesting series of experiments, is the same as that designed in 1911 and since constructed at the Taf Fechan Reservoir, South Wales.

A circular spillway had previously been constructed for the Font Reservoir at Tynemouth, England, about 18 years ago, but, in that instance, the overflowing water first fell into a basin forming a kind of stilling pond, and then over a circular weir formed in masonry. Below the crest of this weir, there was a rapid decrease in diameter to connect with the cylindrical shaft, and the water fell into a deep sump, which, with the stopping in the tunnel immediately up stream, presumably would divert the water. At times of heavy flood, this arrangement must have been hard on the lining of the shaft and tunnel in the vicinity of the sump. In any event, it could have had only a low efficiency.

The author's investigations and his conclusions are of particular interest to the writer, because he was unable to determine the behavior of the Taf Fechan Spillway and Tunnel experimentally. In the Taf Fechan Spillway,

* With Sir Alex Binnie, Son & Deacon, London, England.

the granite crest was 66 ft. in diameter, rounded on its up-stream edge, and the bellmouth was struck with a radius in vertical axial planes of 30 ft., leading into the vertical shaft, 16 ft. in diameter. The desirability of introducing piers on the weir crest to prevent vortex motion and to produce a more uniform depth of flow, was considered; it was deemed inadvisable, however, to allow any projection above the crest that might trap floating débris, such as trees. In the bellmouth, as constructed, there are four granite fins, radial in plan, 9 in. thick, with a maximum projection of 18 in., which tapers off at the top just below the weir crest, and also at the bottom at the junction with the cylindrical shaft. The cross-sectional area of the channel which forms the access to the weir over about two-fifths of its length is so reduced on both sides of the median line as to insure a comparatively low and fairly uniform circumferential velocity. It is anticipated that there will be no great difference between the heads on the weir at the reservoir side and opposite the smallest section of the channel of approach.

The radius of the bend at the foot of the shaft is 15 ft., and there the diameter changes from 16 ft. in the shaft to 13 ft. in the tunnel. To have increased the radius would have added considerably to the cost of construction at this point. With regard to the weir, the writer investigated the flow over sharp-edged circular weirs ranging from 7 to 26 in. in diameter,* and for these sizes the coefficient in $Q = C\,L\,H^{1.42}$ increased from $C = 2.93$ to $C = 3.03$. The fact that the flow varied as $H^{1.42}$ was quite definitely established; subsequently, the index for sharp-edged rectangular and trapezoidal weirs† was also found to be less than 1.50.

Turning now to Table 7, Experiments 2 and 7 are seen to lie well on the curve of Fig. 17. Using these experiments to determine the constant, C, and the index, m, in the formula, $Q = C\,L\,H^m$, it will be found that $C = 2.14$ and $m = 1.34$. That C will have a larger value in the actual spillway, the writer has no doubt, but the difference between 1.34 and the value, 1.50, taken by the author as the basis of comparison for the flow ratio (pages 17-18), is striking, and suggests that instead of this ratio being $\frac{1}{7\,7\,7\,6}$, it should be $\frac{1}{4\,3\,8\,0}$. The increase of C may possibly increase the ratio to $\frac{1}{5\,5\,0\,0}$, but even this would call for a weir of about 40% greater diameter than is proposed, as it does not seem to be feasible to increase the head materially.

B. F. Jakobsen,‡ M. Am. Soc. C. E. (by letter).—The advantage of the ordinary weir spillway lies in the fact that it has a large factor of safety; since the discharge is proportional to $H^{\frac{3}{2}}$, it increases more rapidly than H. The disadvantage of the siphonic spillway lies in the fact that it is nearly independent of H; its advantage, in that it is possible to anticipate a threatened flood by beginning to lower the reservoir.

The proposed spillway type seems to have the disadvantages of the ordinary weir and the siphonic spillways combined. This is illustrated in Fig. 25; when $H = 9.5$ ft., the capacity is about 33 000 sec-ft.; when H is increased

* *Minutes of Proceedings*, Inst. C. E., Vol. CLXXXIV (1911).
† *Loc. cit.*, Vol. CC (1914).
‡ Cons. Engr., Care, Flood Control, Los Angeles, Calif.

to 14 ft., the capacity is increased only 3 per cent. In contrast to this, the weir type of spillway would discharge 80% more under the increased head.

In the writer's opinion, the critical part of the proposed spillway is not the upper, but the lower, part. The discharge capacity of the upper part is proportional to $H^{\frac{3}{2}}$; if the coefficient, c, has been estimated incorrectly, so that the actual flow is only 90% of the estimated flow, this error can be compensated for by a small variation of head, H. For example, an increase of head from 8.0 ft. to 8.6 ft. would accomplish this compensation; but if an error of 10% exists in the lower part, that is, in the shaft, due to an incorrect estimate of the various coefficients involved, a change of head of 0.6 ft. would not help matters. Moreover, the writer believes that the error of calculation is likely to be much greater for the shaft than for the upper part. The arrangement evidently should be such that the knee shown in Fig. 25 will lie well above the crest of the dam.

Great accuracy in determining the spillway capacity is not required, because the maximum flood is not known with any degree of certainty. Probability calculations can give only approximations, even when the records for high floods are fairly accurate, which is seldom the case, and ordinary guessing is likely to be still further from the truth. For earth dams, an excessive spillway capacity should be provided, because the safety of the whole structure is involved. In this case, the ordinary weir spillway seems to be ideal, because it increases its capacity rapidly with H.

Allen Hazen,* M. Am. Soc. C. E. (by letter).—This paper is an interesting account of a novel construction. The writer had the pleasure of inspecting the work that is described while it was under construction, as well as the model of the spillway in operation. He is in no doubt as to the merits of this type of spillway, and commends the excellent experimental work.

The mathematics used to obtain the water down an easy slope are rather appalling; a simpler calculation might answer practical purposes. The mathematics are good mental gymnastics—interesting and possibly helpful.

In 1918, the writer designed a temporary spillway which had enough in common with the one at Davis Bridge to be of interest. This spillway was for the Calaveras Dam of the Spring Valley Water Company of San Francisco, Calif.

It will be remembered that the top of the Calaveras Dam† had slipped into the reservoir, carrying with it the outlet tower and covering with débris the upper end of the outlet conduit. The most pressing problem presented was to draw off the 70 ft. of water then in the reservoir and to find an outlet for further water quantities, for if this were not promptly done, what was left of the great dam would be overtopped and destroyed.

The first remedy was to drive a new and independent tunnel through the adjacent hillside. While this was under way, a second outlet was sought to give added safety and convenience in the subsequent operations. The old outlet conduit, of ample capacity, was buried by the slip under 70 ft. of débris.

* Cons. Engr. (Hazen & Whipple), New York, N. Y.

† *Engineering News-Record*, Vol. 80, p. 679; Vol. 81, p. 1158.

Two small channels through massive concrete at its upper end had been left to serve in draining the bottom of the reservoir; their combined area was 41 sq. ft. A caisson well, 16 ft. in diameter, with walls 2 ft. thick, was sunk through the débris exactly on the site of the old outlet shaft to the foundation, which proved to be intact. A short connecting tunnel was then driven through the débris, connecting the bottom of this well with the previously mentioned openings in the head of the old conduit. These openings limited the capacity. As first built, the caisson well was 80 ft. deep. It was supposed that water would flow over its edge and fall into the well and accumulate in it, but always flowing according to the head through the openings to the old conduit. The conduit itself had capacity to carry more water than could be taken to it. It was expected that between 1 600 and 2 000 cu. ft. of water per sec. could be discharged. Under existing conditions, with many feet of free-board, there was space to hold temporarily the run-off from a flood without overtopping the dam, and this outlet, or the new one, or both together, would carry it off gradually.

In practical operation, the spillway worked admirably, but not exactly as expected. When the water began to go over its edge, it broke into a fine spray, and, falling with a high velocity, carried air downward with such force that the current of air going through the old conduit and coming out hundreds of feet below, was greater than a man could face.

In the five years of operation, the water has never been deep enough to submerge the inlet completely, and the downward flow of water has always been accompanied by a flow of air. When the writer last saw it, after some fairly large floods, the concrete in the narrow passages at the bottom showed no erosion from use. This was in contrast with earlier experience, before the slip of the dam, when, under much lower heads, with solid water, there had been considerable punishment of the very heavy and hard concrete in these same narrow channels.

As a result of this experience, the writer was led to believe that the best design for such a spillway is in the form of a gigantic air-pump. The air, intimately mixed with the water, gives it elasticity and prevents the making and breaking of vacuum with accompanying shock, which is so destructive to outlet structures where solid water flows at high velocities.

G. A. M. Elliott, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, has since raised the flow line of the reservoir and the shaft of the spillway, so that the drop is now 100 ft.

It may also be of interest to note that the principal spillway of Lake Chabot of the East Bay Water Company is a tunnel. This tunnel spillway, built in 1889, is 1 846 ft. long, horseshoe-shaped, and 10 ft. wide, and 10 ft. high throughout. The slope is steeper at the upper end, but notwithstanding this, the limiting point of its capacity is undoubtedly at its entrance. This entrance originally had square corners of cut stone; these were afterward cut off and replaced with concrete, rounded to ease the entrance and increase its capacity, now estimated at 3 000 cu. ft. per sec., more or less.

It may be noted that shortly after this Calaveras experience, the writer designed a spillway to operate in much the same manner for the San Pablo

Dam near Berkeley, Calif., but as this was a new design, some developments and improvements were possible.

A preliminary design was also made in 1918 for a permanent spillway for the Calaveras Dam. This was a study only, and the plan was not adopted. This design was intended to avoid permitting large quantities of water to drop 200 ft. or more on rock so soft as to be eroded by water at high velocities. The proposed outlet tunnel was about 2 700 ft. long with a drop of about 350 ft., the tunnel being 18 ft. in diameter, except that it was enlarged in its upper end where the full velocity could not be developed. This tunnel was intended to carry a possible flood of 20 000 cu. ft. per sec. The edge of the circular inlet basin was 120 ft. in diameter. If it should ever be built, it would no doubt result in a very considerable change in the landscape near the outlet of the tunnel, but as this is far away from the dam, in wild country, and separated by large hills of hard rock, no danger to the dam would result.

All the experience with tunnel spillways so far has been favorable, and the writer believes that their use will prove advantageous in many situations that render open spillways of the usual construction difficult or expensive.

KARL R. KENNISON,* M. AM. SOC. C. E. (by letter).—A most significant fact in connection with a shaft spillway for an earth dam, such as that at the Davis Bridge site, is the positive limitation of the capacity, due to the fact that the critical discharge is throttled by the shaft and tunnel and is independent of the spillway crest.

In this case, the rating curves of the model indicate that the limit will be about 34 000 cu. ft. per sec., or 185 cu. ft. per sec. per sq. mile of total drainage area and 220 cu. ft. per sec. per sq. mile of area below the Somerset Reservoir. A discharge at a higher rate than this could occur only by overtopping of the earth dam, which would cause failure. Hence, it would be most interesting from the standpoint of design to have recorded the character of the water-shed and its run-off and the computed effect of the reservoir surface areas in reducing flood peaks.

This limit of the discharge to a rate which cannot be exceeded under any consideration presumably has been conservatively applied in this case, when the probable effect of the reservoir area is taken into account. The writer calls attention to it because of its importance in the general problem of shaft spillways.

In determining this fixed limit of the capacity in designing such a spillway, the question naturally arises as to the cost of increasing it. Can the discharge be increased, say, 10 or 20 or 30% at small expense in order to obtain added safety? In general, there are two ways of doing this, the feasibility of which would have to be determined for the case in hand:

1.—By a considerable enlargement of the short vertical shaft at small added cost so that high velocities can begin in the straight tunnel with a high entrance coefficient.

* Cons. Engr., Boston, Mass.

2.—By a location and design of the discharge end of the tunnel so as to recover as much as possible of the velocity head; this method of increasing the discharge would be particularly effective under a low total head.

The details of the design of the spillway crest itself, since they have no important bearing on the discharge capacity, would naturally be governed largely by practical considerations involved in the operation of stop-logs or gates, or the care of ice, etc. The exact shape of the crest to pass the given discharge at a low head is of little consequence.

One of the reasons advanced for not building the ordinary type of spillway is that the steep slopes of the canyon walls would make the construction expensive on account of excessive excavation. This is also an argument against building the crest around the shaft in the form of a circle of such large diameter. Where the canyon walls are steep, the spillway crest could be built in straight sections along the contour of the hill running into a smooth bell-mouth shaft entrance. This would result in a cheaper concrete structure for the crest and for the stop-log bridge and handling apparatus, and yet it could easily be made to produce the same rating curve for the discharge of floods, and would involve no great difficulties in hydraulic design.

Practically none of the head saved by the exact circular crest is applied to increasing the discharge.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).—The author is to be commended for contributing this paper so replete with detail calculations; likewise, the New England Company for undertaking the interesting model experiments.

The writer has always favored obtaining information from models, and believes that benefit would accrue to both owners and constructors by making greater use of experimental research. Models should be constructed to as large a scale as circumstances will permit, and results obtained by observation and measurement should not be extended beyond the limits of experimentation with the model. Water, air, and barometric pressure are the same in experiment and practice, but friction, eddies, and similar phenomena will not be directly proportional to the model scale adopted. Corrections for model scale will be difficult to apply accurately.

The amount of air drawn in by the spillway model will not be relatively proportional to the amount entrained by the water in the actual spillway, as the velocities and suction effects of water entering the shaft are so different in the two cases. The suction effect of a flow of water into a funnel that is transferred into a shaft will change rapidly with velocity and depth of water. As the depth and velocity increase, the entrained particles of air will become rarefied, that is, a partial vacuum will be produced. This will be changed later to positive pressure (compression), as velocity is lost by friction in the shaft and tunnel. At some stage of flow—probably not indicated in the model—this change of air from vacuum to compression in the shaft may set up a vibratory action. The rock through which the shaft is excavated may be massive enough to sup-

* Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., New York, N. Y.

press all signs of vibration, but it will exist and may affect the lining, should any part become loosened.

Ice cakes or logs, carried into the spillway, will pass into the shaft at high velocity, and are likely to strike heavily against the lining of the bend. Possibly some precaution, not referred to in the paper, is being taken to prevent trouble from this cause. A compound curve for the shaftway bend would have been better than the circular curve, but, perhaps, the conditions at the site prevented this refinement, as they did the use of a longer radius for the curve.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—A brief visit to the Davis Bridge Dam in the fall of 1923 led to a feeling of surprise because the unfinished up-stream and down-stream slopes were not terraced or rolled and were very steep, the material being dumped from trains. The relation of the ends of the middle or pool area to the mountain sides was not clear. The author's paper is on the spillway, at that time partly established, and two of its features seem to invite mention to the exclusion of other phases of the work.

For a considerable part of its length, the axis of the spillway is vertical. The tunnel-curve section and somewhat more is in the solid rock of the hillside. If the surrounding material other than rock in place were to be taken away, the upper part of the structure would be in the air and unsupported except by the wall of the shaft. In appearance, then, the spillway would closely resemble the author's Fig. 3. What provision was made to support the outer edge or lip of the spillway? Does it rest on a circular wall of masonry or concrete 160 ft. in diameter springing from the rock below and of the variable height necessary to meet the horizontal plane which defines the overflow? If so, what are the dimensions of that wall? How was the space between that outer wall and the shaft wall filled or treated? Gravitation and the quality of the material of the adjacent formations have helped to shape the hillside slopes of that valley; changes in slope or material will not exempt either from the active forces that have prevailed in the past.

In Vermont, not more than sixty miles north of the Davis Bridge Dam, and on ground about 700 ft. lower, there are several large shafts open to the sky and of depths comparable with that of the spillway shaft. Many tons of coal are lowered into those shafts and burned to melt the ice formed in the winter season, to make the frozen rock "lift better", and to enable the workmen to be more comfortable in summer and winter. The temperature of the rock is very low. There are places in that part of the State where ground or well water registers less than 40° Fahr. in the summer season. In winter, the cold air constantly descends and upward currents carry away increments of heat; in summer, this process is not reversed. Little heat from the sun finds its way to the bottom at any season. Those who have been thrilled by a view of the American Fall from Inspiration Point and who, from the same viewpoint a week later, were unable to see any water at that Fall, will realize more fully the effects of continued low temperature in the latitude of Niagara.

* Civ. and Hydr. Engr., New York, N. Y.

EUGENE E. HALMOS,* M. AM. SOC. C. E. (by letter).—The hydraulic and structural design of shaft spillways should receive serious study by engineers interested in hydro-electric developments. The author deserves a great deal of credit for bringing this subject before the profession.

The writer believes that, although this method of disposing of flood waters seldom is economical on wide streams and, especially, in connection with earthen dams, it will be frequently used on narrow streams flowing in deep valleys. In the design of a number of power plants for hydro-electric projects on both Eastern and Western streams, necessitating the construction of high dams built in narrow canyons, he has found, with few exceptions, that the shaft spillway presented the cheapest and most satisfactory means of flood disposal. One or more diversion tunnels built to take care of the river flow during construction assures not only reasonably continuous building operations, but makes it unnecessary to break up the work into small sections. One single coffer-dam can be provided for the whole dam with a corresponding important reduction in cost. With proper foresight in the design, such diversion tunnel or tunnels can be given dimensions to correspond to the requirement of the shaft spillway, thereby incorporating in the permanent structure a piece of work needed for construction purposes.

The writer believes that the method used at Davis Bridge for accelerating the water into the shaft by a "free" fall on the converging apron of the spillway is open to criticism. In his opinion, better results can be obtained by building up the column of water above the shaft opening to a height which will give the necessary accelerating head. This is easily accomplished by the construction of a standard spillway dam and by excavating the enclosed area to a depth and to slopes determined by the static head needed to produce the velocity required in the shaft. By this method, full advantage can be taken of the pressure represented by the weight of the atmosphere, thereby considerably diminishing the necessary depth of the water above the opening and correspondingly reducing the volume of the excavation. The tests described by the author plainly indicate, as it is also remarked in the paper, that the maximum capacity of the spillway was reached when the shaft acted as an orifice. There seems to be no reason, therefore, to design the shaft for a less efficient condition.

The siphon or draft-tube action which is inherent to the upper part of a vertical pipe flowing at capacity, and which, curiously enough, is mentioned in the paper as unexplained and as not having been anticipated, will result at Davis Dam, as demonstrated by the tests, in sucking air into the shaft and causing tremor and disturbed flow at all discharges except those above the designed capacity.

Producing the accelerating head by pressure, as proposed by the writer, will prevent air from entering the conduit for a considerable range of discharge volume. It is even possible to obtain this condition for all discharges by dividing the shaft into several conduits and controlling their inlets with gates. The flow then will be smooth and less likely to injure the concrete

* Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

lining of the shaft and of the tunnel than the eddying, surging flow resulting from the freely falling jet.

Charles W. Comstock,* M. Am. Soc. C. E.—The speaker's first impression on reading this interesting hydraulic study was that 27 000 sec-ft. seemed a very large spillway capacity to provide for floods from a drainage area of only 180 sq. miles. With this in mind, he made a calculation to determine the necessary time for the water surface to rise from the elevation of the spillway sill to a level about 8 ft. higher.

The spillway without piers or flash-boards was used; the area of the reservoir surface was taken as 2 000 acres (assumed constant throughout the rise); a constant inflow of 27 000 sec-ft. was assumed; and the author's coefficient for the weir was accepted as correct. The calculation showed that about 21.5 hours would be required for the water to reach a depth of 7.84 ft. above the spillway sill. Theoretically, the time required to reach a depth of 8 ft., at which the outflow would equal the inflow, would be infinite. During this time the inflow would be approximately 48 000 acre-ft., of which about one-third would be stored, the remainder passing through the spillway. This volume of water is equivalent to a run-off of 5 in. from an area of 180 sq. miles, and would probably represent a precipitation of 7 or 8 in., certainly an enormous, although not an unprecedented, rainfall.

The amount of money which may properly be expended to provide against remote contingencies is a matter of judgment, in which the engineer is responsible only to his client, and concerning which he should not be criticized because others hold different views. The most that can be done in discussing such a point is to bring out as many expressions of opinion as possible, leaving to each man who may encounter similar problems in the future the privilege of making a decision for himself.

The second point which seems to call for comment is one which is not peculiar to the shaft type of spillway, namely, the use of flash-boards. The spillway is presumably designed with the same care that is given to any other part of the work, and embodies the engineer's best judgment as to margin of safety. To obstruct it with flash-boards for the sake of increased storage capacity is analogous to providing a steam boiler with a safety valve and then tying it down to permit higher pressure, on the assumption that the attendant will watch the gauge and release the valve if the danger point approaches. Safety valves, spillways, and all types of automatic protective devices are intended to provide against the absence, inattention, or disability of attendants, as well as other unforeseen happenings, and the speaker believes that the engineer, having used his best judgment in determining the proper margin of safety, should have the courage of his convictions and not permit encroachment on that margin, however attractive and desirable an increase of storage, speed, or pressure may seem.

There has recently come to the speaker's attention the case of a reservoir located near the head-waters of a stream where a regular practice of using

* Engr., Dwight P. Robinson & Co., Inc., New York, N. Y.

flash-boards in the spillway was in force. Some time ago, following heavy rains, it became necessary to remove the flash-boards. The dam in question suffered no damage, but during this same flood period an earth dam farther down the stream was over-topped and destroyed. The owners of the upper reservoir are now being sued for heavy damages, the charge being that the excess water released by removal of the flash-boards, added to the already swollen stream, caused the damage below. Whether or not this is true, or whether the lower dam might have been topped by the natural flood without the addition of reservoir water, it is certain that the upper reservoir would not have been open to the charge and under the necessity of defending a costly lawsuit had the spillway been clear and unobstructed at all times.

F. W. Scheidenhelm,* M. Am. Soc. C. E.—This paper is one which, however much it is valued at the time of its presentation, will be most deeply appreciated by engineers who later refer to it in connection with analogous problems. It contains a concise, yet clear, explanation of the theory, design, and experimentation pertaining to the subject of shaft spillways. Moreover, it possesses the exceptional merit of not leaving the reader in doubt as to pertinent assumptions or conditions affecting this phase or that.

Several requisites are involved in such a contribution as that of the author. As a matter of course, there is first the detailed design, supplemented in the present instance by experimental study. Generally speaking, it is only for an important project that such extensive and expensive theoretical and experimental investigation is involved or can be afforded. Consequently, it is necessary that the appropriate executives consent to the publication of the results of the work. In the present instance the profession should duly appreciate the willingness of the New England Power Company and the J. G. White Engineering Corporation that this paper should be prepared and the results made public by a responsible engineer as a part of his due contribution to engineering knowledge. Ordinarily at the conclusion of the design the office records are not in shape for publication. Granting the requisite ability, there is necessary the willingness on the part of an author to devote a great deal of time and energy to the preparation of the material, for, when once the engineering work is done, he has a strong temptation to place the subject-matter on a shelf, inaccessible to fellow engineers.

The paper sets forth the evolution of this particular design and its experimental demonstration and adjustment in such detail as to create a longing for more, namely, for a demonstration in full-size operation. This is something which, presumably, only Nature can afford; but inasmuch as it appears probable that this shaft spillway will be required from time to time to take care of floods which are not dwarfed into insignificance by the capacity of the reservoir to absorb flood peaks, it is to be hoped that the results of such occurrences will be accurately observed and similarly published. This hope finds the greater basis in the fact that this spillway serves one of a number of hydro-electric power plants along a river which is being developed intensively. Therefore, one may anticipate close hydraulic supervision by a capable

* Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

technical staff that will record both the pertinent reservoir levels and the corresponding discharges through the shaft spillway.

The determination of appropriate dimensions, of frictional and flow factors for homologous models to be used in experimentation, is a complicated and important matter. One of the incidental features of the paper, but one which the speaker rates as of considerable importance, is the exposition covering the design and choice of factors for the models which were tested.

The willingness to subject surfaces of concrete to contact with water flowing at so high a velocity as 68 ft. per sec. (average), requires considerable courage. There are few, if any, precedents by way of structures intended to endure similar velocities. The speaker would be surprised if, on the occurrence of velocities approximating the anticipated maximum, any considerable roughness or spalling in the concrete did not result in material local erosion, extending perhaps through the lining or even into the rock backing. Fortunately the conditions are such that no extensive harm could result and the waterway, having performed its duty of taking care of a critical flood, could be patched or otherwise repaired. Performance in this respect for velocities beginning as low as 20 ft. per sec., constitutes one of the important features which should be made public from time to time, in order to complete the story which the author has so admirably begun—and, in fact, has carried well along toward its concluding chapters.

The paper as a whole leaves the speaker with the impression that the design is in exceptional measure based on rational analysis instead of mere judgment.

H. K. Barrows,* M. Am. Soc. C. E. (by letter).—There are many interesting features in the design and construction of the Davis Bridge Spillway and Dam and the author is certainly to be commended for his comprehensive and detailed presentation of the methods used in experimentation and in the design of the shaft spillway. In adding to the discussion of this paper, the writer will present the results of further experimental work on the model spillway made at the Massachusetts Institute of Technology in 1923.

Necessary Capacity of Spillway.—The record of flow of the Deerfield River at Charlemont, Mass., from 1913 to date (1924), affords a fairly good basis for an estimate of the flood flow to be expected at Whitingham, Vt.

For the ten years of record at Charlemont (for an area of 362 sq. miles, less 30 sq. miles at Somerset Reservoir, or 332 sq. miles net), the "average yearly flood" is 10 900 sec-ft., or a flood coefficient of $C = 105$, in the formula, $Q = C A^{0.8}$, as used by Weston E. Fuller, M. Am. Soc. C. E.† Applying this coefficient to the area above Whitingham (184 sq. miles, including for safety the 30 sq. miles at Somerset Reservoir) gives an average yearly flood to be expected at Whitingham of about 6 800 sec-ft.

Using $T = 1\,000$ years, there results a maximum daily flow of 23 100 sec-ft., with a crest flow of 33 000 sec-ft. (or 3.4 times the average flood); using $T = 10\,000$ years these quantities become 28 600 and 40 500 sec-ft., respectively (or 4.2 times the average flood).

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† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564 *et seq.*

At the vicinity of the spillway level of the Davis Bridge Dam (Elevation 1 386), the water area of the reservoir is about 2 200 acres. Assuming a parabolic distribution of flow during the flood day, and starting with water at spillway level (Elevation 1 386.0), detailed studies give the hydraulic data shown in Table 10.

TABLE 10.—Hydraulic Data at Davis Bridge Dam for 1 000-Year and 10 000-Year Floods.

Item.	A. 1 000-year flood.	B. 10 000-year flood.
Flow into reservoir for 24 hours of maximum flood day, in second-feet	23 100	28 600
Maximum crest flow into reservoir during flood day, in second-feet	33 000	40 500
Maximum elevation of water surface on spillway, in feet	1 393.7	1 395.0
Corresponding head on spillway, in feet	7.7	9.0
Maximum discharge over spillway, in second-feet	27 400	34 400

In the foregoing studies, a value of $C = 2.85$ in the formula, $Q = C\,L\,H^{\frac{3}{2}}$, for the spillway was used. As a matter of fact the capacity of the tunnel limits the possible discharge of the spillway to about 30 000 sec-ft. Due to this limitation the elevation reached by the water surface at the spillway during the 10 000-year flood would have been increased about 0.6 ft., or to about 9.6 ft. Even with the foregoing severe assumption as to possible flood, as the top of the earth dam is at Elevation 1 400, there would still be a free-board of about 4.4 ft.

The highest observed flow at Charlemont was about 45 000 sec-ft. at about 11: 00 P. M. on July 8, 1915. For the 24 hours from noon, July 8, to noon, July 9, the discharge averaged 20 500 sec-ft. For the two days, July 8 and 9, the rainfall at Somerset totaled about 5 in. (following several days of rainfall subsequent to June 30, which amounted to about 3 in.). Assuming the same flow per unit of area at Davis Bridge as at Charlemont (namely, 136 sec-ft. at the crest and 62 sec-ft. as the average for 24 hours from each of the 332 sq. miles below Somerset Reservoir, the latter not contributing to this flood), the 1915 flood would have given a crest discharge at Davis Bridge of about 21 000 sec-ft. and an average discharge for 24 hours of about 9 500 sec-ft. The foregoing 24-hour discharge for the 1 000-year flood is, therefore, about 2.5 times that of the 1915 flood; the assumed 10 000-year flood is about 3 times that of 1915.

Experiments on Model Spillway at Massachusetts Institute of Technology.—Through the courtesy of the engineers of the Power Construction Company, the model spillway tested at Whitingham and at the Worcester Polytechnic Institute, was shipped to the Hydraulic Laboratory of the Massachusetts Institute of Technology late in 1922, so that it was possible to make some experiments* on the capacity of this spillway and tunnel under conditions differing somewhat from those for either of the previous tests.

* These later experiments were made by Messrs. E. S. Birkenwald and F. R. Morgan as thesis work for the Master's Degree and were performed in the spring of 1923 under the direction of the writer.

The general arrangement of the spillway and testing apparatus is shown in sectional elevation on Fig. 26. The model was set in the 10-ft. canal in the basement of the Hydraulic Laboratory, about 22 ft. up stream from the end of this canal, where ordinarily water passes over a 10-ft. weir. This weir, however, in these tests was tightly closed by a bulkhead in order to maintain the water level in the canal at any desired height with reference to the spillway model. About 29 ft. up stream from the model, a new bulkhead was constructed in which was placed a 4-ft. standard weir with end contractions, the weir crest being formed by the beveled edge of a 6-in. steel angle and the end contractions by beveled steel plates. Readings of head on the weir were made by a hook-gauge in a stilling-box placed about 6½ ft. up stream from the weir, at the side of the canal.

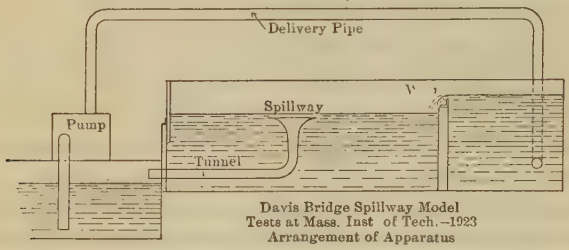

FIG. 26.

A wooden deck was built in imitation of the bench in the cliff which partly surrounds the full-sized spillway. The approach was shaped to the spiral curve adopted for the design, using sheets of galvanized iron. At Davis Bridge there will be a transition curve of 30-ft. radius rising from this bench to the lip of the spillway. In the model this curve was built of cement mortar moulded into place by a wooden templet of 10-in. radius.

For the model tunnel ordinary stovepipe 7½ in. in diameter was used. Piezometers were placed in the tunnel at three points (Nos. 4, 5, and 6, Fig. 27) to enable pressure readings and loss of head determinations to be made under various conditions of flow. These connections were arranged with gauges consisting of glass U-tubes partly filled with water. Piezometer connections were also made at Points Nos. 1, 2, and 3 (Fig. 27) in the model spillway—the same points that were used for this purpose in the previous sets of experiments.

The model was mounted on a wooden frame and connected with the pipe tunnel by a special fitting. The joint between the model and this fitting was caulked with oakum, plastered with cement, and held securely in place by eight bolts passing through the flange of the model and the planks on which it set. For the pipe ordinary slip-joints were used, covered with elastic cement, and held in place by galvanized iron sleeves tightened with bolts, all seams in the pipe being closed with elastic cement or solder. The pipe was supported by small wooden horses and weighted down by a cast-iron pipe slung beneath it. The model was weighted down by about 600 lb. of paving material piled on the supporting frame, and anchored against horizontal movement by timbers spiked across the canal.

Ample water capacity was supplied by a duplex pump with a capacity of about 3 sec-ft. and auxiliary pumps to give a combined capacity of about 6 sec-ft. It was found during the tests, however, that the model could be drowned out by the larger pump alone. Water was delivered at the bottom of the canal about 25 ft. above the weir, giving a stilling-bay approximately 7½ ft. deep, 10 ft. wide, and 25 ft. long, and resulting in a calm and smooth water surface for hook-gauge readings. Runs were made both with and without the flash-board piers, during which measurements of head on the model, pressures at the various piezometers, and discharges were obtained.

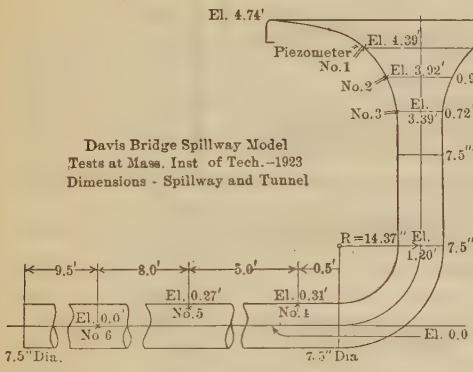

FIG. 27.

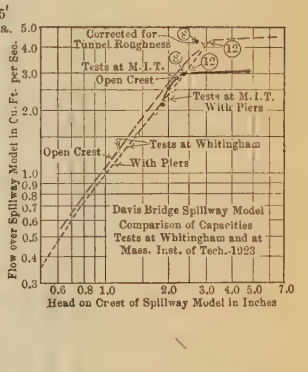

FIG. 28.

There was a slight leakage around the ends of the new bulkhead containing the weir. Careful measurements in several tests showed this to be about 0.04 sec-ft., which amount was used as a correction factor.

Summary of Results.—In Table 11 is given a summary of the more important observed data for those experimental series which are the most typical and representative. Each series is based on three or more individual sets of readings.

Capacity Curves.—On Fig. 28 are given the capacity curves resulting from these experiments plotted with reference to the head on the model and, for comparison, similar curves obtained in the experiments at Davis Bridge, as shown on Fig. 17. As will be noted, the limit in capacity of the model tunnel in these experiments was reached with a lower discharge than at Davis Bridge, due to the greater coefficient of roughness for the tunnel.

This limit in capacity or "drowning out" of the model spillway occurred at a discharge of substantially 3.0 sec-ft., for which the head on the model spillway crest was about 2.25 in. for the open-crest arrangement and about 2.35 in. for the crest arrangement with piers. Below this critical point, in the tests in which piers were used, the discharge curves are nearly the same for both sets of experiments; for the open-crest type, the experiments at Whitingham show a little greater spillway capacity than those at the Massachusetts Institute of Technology.

Hydraulic Gradient in Model Tunnel.—Pressure readings for piezometers in the tunnel (Nos. 4 to 6 in Table 11) were rather variable and uncertain for

TABLE 11.—Summary of Experiments, Davis Bridge Model Spillway, Massachusetts Institute of Technology, 1923.

Date.	Series.	Pressure Head, in Feet, at Piezometers Referred to Center Line of Tunnel as Datum.					Discharge, in second-feet.	Head on Crest of Model Spillway.		Computed value of C, for spillway model *.	Remarks.
		No. 2.	No. 3.	No. 4.	No. 5.	No. 6.		Inches.	Feet.		
April 28, 1923, (open crest.)	5	3.95	3.44	0.370	0.345	0.091	2.05	1.80	0.150	2.54	.....
	6	4.29	3.94	1.162	1.020	0.362	2.65	2.03	0.168	2.74	.....
	7	4.31	3.95	1.396	1.074	0.460	2.87	2.16	0.179	2.70	Tunnel nearly full.
	8	4.36	3.97	1.416	1.138	0.560	2.99	2.34	0.195	2.48	Tunnel completely filled.
	9	4.40	3.99	1.409	1.156	0.695	3.07	4.89	0.406	....	.....
May 1, 1923, (with flashboard piers.)	1	3.90	3.36	0.320	0.278	0.008	1.53	1.43	0.119	2.95	.....
	8	3.91	3.37	0.320	0.292	0.016	1.78	1.60	0.133	2.93	.....
	9	3.92	3.38	0.351	0.312	0.042	1.97	1.73	0.144	2·87	.....
	10	3.895	3.37	0.454	0.409	0.217	2.23	1.87	0.156	2.89	.....
	11	3.895	3.39	0.767	1.033	0.400	2.68	2.16	0.179	2.85	.....
	12	4.25	3.85	1.427	1.155	0.581	3.02	2.45	0.205	2.59	Tunnel completely filled.
May 3, 1923, (with alternate piers removed.)	1	3.90	3.36	0.587	0.955	0.376	2.48	1.97	0.164	2.84	.....

* Using length of crest of 13.96 ft. for open crest, 12.55 ft. for crest with piers, and 13.25 for crest with alternate piers removed.

a discharge of less than 2.5 sec-ft. A study of the gauge readings and a plot of the hydraulic gradients between Piezometer No. 4 and the end of the model tunnel (see Fig. 27) resulted as shown in Table 12.

TABLE 12.—CONSTANTS FOR TUNNEL DISCHARGE.

Discharge, in second-feet.	Hydraulic gradient in tunnel.	f, in $\frac{fl}{d}\frac{v^2}{2g}$.	C, in $C\sqrt{RS}$
2.5	0.045	0.267	98
3.0	0.062	0.264	99

Effect of Model Tunnel on Spillway Capacity.—The model tunnel was 7½ in. in diameter, its hydraulic radius, 0.156 ft., and its length, 26.3 ft., differing slightly from the exact relative dimensions $\left(\frac{1}{36}\right)$ of the full-sized tunnel which would have been, hydraulic radius, 0.149 ft., and length, 26.6 ft. On the other hand, the difference in elevation between the spillway crest and the center of the tunnel at the exit was 4.74 ft. for the model instead of 4.78 ft., as given on Fig. 2, showing a slight deficiency of discharge.

For the model tunnel, as noted by the author, $\frac{l}{C^2}$ should be $\frac{1}{36} \times \frac{957}{136.5^2}$ $= 0.001425$. The value of C^2, if $l = 26.3$ (as at the Massachusetts Institute of Technology) would be 18 400, and C should have been 136, whereas the test value was 99. As the discharge would vary approximately directly as C, the maximum discharge through the model tunnel would have been $3.0 \times \frac{136}{99}$ $= 4.1$ sec-ft., if the model tunnel had been of a length and a degree of smoothness comparable to that assumed for the full-sized tunnel.

This greater discharge also would have increased the head on the model spillway to the following amounts:

(*a*).—Open crest:

$$4.1 = 2.70 \times 13.96\, H^{\frac{3}{2}}$$
$$H = 0.222 \text{ ft.} = 2.66 \text{ in.}$$

(*b*).—With piers:

$$4.1 = 2.85 \times 12.55 \times H^{\frac{3}{2}}$$
$$H = 0.236 \text{ ft.} = 2.84 \text{ in.}$$

The computed values of discharge and head on the spillway for Series 8 and 12 are shown on Fig. 28, and accord reasonably well with those of the previous experiments at Whitingham, in which the tunnel was made as smooth as possible, with a length of 17 ft. from the bend to the end of the tunnel (as compared with 23 ft. at the Massachusetts Institute of Technology), with an assumed coefficient of roughness, n, of 0.009 (which was not verified, however, by the piezometer readings).

The corresponding capacity of the full-sized spillway and tunnel, based on the Massachusetts Institute of Technology experiments, is, therefore, approximately as follows:

$$Q = 4.1 \times \frac{398}{0.307} \times \sqrt{\frac{5.36}{0.156}} = 31\,400 \text{ cu. ft. per sec.}$$

The head reached on the spillway crest would be:

(*a*).—Open crest:

$$2.75 \times \frac{36}{12} = 8.25 \text{ ft.}$$

(*b*).—With piers:

$$2.87 \times \frac{36}{12} = 8.61 \text{ ft.}$$

By reference to Fig. 25, it will be seen that according to the author's conclusions the ultimate capacity of the spillway tunnel (with piers) was reached at a discharge of about 33 500 sec-ft., with a head on the spillway of about 9.3 ft., as compared to 31 400 sec-ft. and 8.6 ft. based on the Massachusetts Institute of Technology experiments, indicating a slightly smaller tunnel capacity for the latter.

Based on the results of the Massachusetts Institute of Technology experiments alone, for the spillway with piers, the limitation in discharge capacity of the spillway and tunnel of 31 400 sec-ft. would result in a maximum elevation of water surface at the spillway for the 10 000-year flood of about 1 395.3, instead of 1 395.0 based on the previous experimental work. Considering the different methods used in these experiments, however, this is a fairly good agreement.

The model spillway is again at the Hydraulic Laboratory of the Institute, and it is planned to experiment further with it—particularly to make a more extensive study of the losses of head between different sections of the spillway and tunnel.

Use of Tunnel Type of Spillway.—The Davis Bridge spillway is a novel type of construction, the use of which evidently depends on:

1.—A sufficient head at the dam to permit the use of a tunnel section of reasonable size and cost.

2.—A limit in the size of the drainage area and, therefore, of the necessary flood capacity to be handled, due to the size of tunnel section required. Even here, however, the same difficulty would apply in a measure with other types of spillway.

3.—A large water area for the reservoir such that flood peaks may be absorbed by storage above spillway level, thus greatly lessening the maximum capacity of spillway required and reducing the flow to be taken by the spillway to practically an average daily (24-hour) flow instead of, perhaps, that for the maximum hour of the day.

Another field of use for this type of spillway is in the case of arch dams, where it may be impossible to provide a spillway outlet auxiliary to the dam and where the section of the arch dam is not adapted to use as a spillway.

It is likely, too, that the conditions favoring the use of an arch dam, in general, would favor the use of the tunnel spillway.

The behavior of the Davis Bridge spillway and tunnel under conditions of flood flow will be awaited with interest and should furnish valuable data in a field of hydraulics about which information is now very meager. The loss of head at the bend between the vertical shaft and the tunnel is a particularly uncertain element entering into the estimation of the tunnel capacity, both on account of the size of the cross-section and the high velocities, and it is especially to be hoped that this may be determined by measurement.

E. W. Lane,* Assoc. M. Am. Soc. C. E. (by letter).—The author deserves the thanks of the profession for bringing to its attention the shaft spillway, which promises to be a useful device under certain conditions.

There are, in general, four phases which may be considered in discussing the shaft spillway, as expressed in the following questions: (1) What are the advantages and disadvantages of the shaft spillway? (2) What are the advantages and disadvantages of the particular shape used at Davis Bridge? (3) Could any other shape have been used to greater advantage? (4) Is the author's analysis, as developed mathematically and by experiment, correct in all particulars?

The spillway of an earth dam, especially one of the magnitude and location of that at Davis Bridge, is of vital importance, and a comparatively untried type should not be used without a thorough study, not only of the theory, but also of the possible, even though improbable, things which may happen to it. Taking this viewpoint, what are the advantages and disadvantages of the shaft spillway? The greatest advantage—in fact, the only one which seems prominent—is its low cost under such conditions as existed at Davis Bridge.

The danger of clogging from trees and drift brought down by a flood, especially at the bend, is probably the greatest disadvantage of the shaft spillway, and would seem to limit its use to large discharges. For example, recently in designing a spillway for a small earth dam, the writer found that the shaft type possessed great economy although the danger of clogging precluded it. The bend should be of sufficient radius to pass long logs without wedging; the combination of diameter of shaft and radius of bend of the Taf Fechan Spillway mentioned by Mr. Gourley is near the lower limit of safety in this regard.

The probability of clogging is considerably greater than in the case of the conduit of a retarding basin, for the latter soon becomes submerged, and the drift is not drawn in; furthermore, the retarding basin conduit is usually straight. Yet, this contingency controlled, to a large extent, the selection of the size of the outlet conduits of three of the five basins of the Miami Conservancy District. The smallest discharge of any of these conduits, when the basin is filled to spillway level, is 8 800 sec-ft.

In cold climates there would be a possibility of such a spillway filling with ice. At Davis Bridge, as this formation would be ring-shaped, it would tend to hold itself in place by arch action. Furthermore, the air currents which

* Asst. Engr., The Dayton Morgan Eng. Co., Pueblo, Colo.

would naturally form in the chimney-like tunnel would increase the tendency to freeze.

The disadvantage of having what is practically a maximum discharge has already been noted by Mr. Kennison. When the shaft spillway is flooded out, the discharge increases very little with an increase of head, probably even less in the actual spillway than is indicated by Fig. 25, for the vacuum in the shaft would tend to exceed 33 ft., and as it cannot, the discharge would be somewhat reduced from that indicated by the model. What would actually take place under these conditions is difficult to visualize, but it is likely that some vibration would be set up; because of the great mass of rock surrounding the tunnel, this would do little damage except possibly to the tunnel lining.

The disadvantage of having a maximum discharge could be avoided if the shaft were designed so that this value would not be reached until the water in the basin rose to, or above, the elevation of the top of the earth section; but if sufficiently conservative predictions were used in determining the size of the maximum flood, the disadvantage of a limiting discharge would be more apparent than real. Every form of spillway for an earth dam has a limiting discharge, which is the capacity of the spillway when the water stands at the top of the dam.

Except for the vibrations that it might produce, the air drawn in to the spillway would be no disadvantage until it reduced the discharge capacity of the shaft to such an extent that the water backed up considerably above the level of the spillway crest. It is doubtful whether this would ever occur, for with the water backed up in this manner, the conditions causing the air to be drawn in would be largely removed, thus relieving the situation before it became critical, and, therefore, such a backing up would not result.

The foregoing discussion applies to shaft spillways in general; in addition, the form described in the paper seems to have other disadvantages which might not exist in all cases. The low head used on the spillway greatly increases the probability of clogging by drift, on account of the relatively low velocities of flow over the crest. The wide flat crest has a similar effect. If a tree with roots and branches reaches the crest of a high ogee dam and the water is deep enough, the root end is carried over and, after passing the crest, falls and raises the other end out of the water sufficiently to enable it to pass over also. With a crest width of 38 ft., and depths of water of 5 to 7 ft., at velocities of only 12 to 15 ft. per sec., such a possibility is doubtful.

This tendency of drift to catch on the crest is further augmented by the piers, and particularly by the bridge resting on their tops. Judging from the size of the model, there is a clear space of only 12 ft. between the weir crest and the bridge, which would be inadequate to pass trees. The low velocities and shallow depths of the approach channel add another hazard.

It might be argued that in lakes such as that formed by the Davis Bridge Dam, floating drift could be easily kept from the spillway. This no doubt is true in ordinary floods, but it will never be possible to determine the adequacy of the equipment for large floods until they come, and then it will be too late to remedy any defect. Even a watchman permanently stationed at

the dam might have his communications cut off by a large flood, and be unable to summon aid. At the Austin, Tex., dam, an immense raft has formed, covering the entire river for some distance above the dam with drift that has been prevented from passing over because of the gates and piers on the crest.

Some abrasion may occur at the bend in the shaft, due to the impact of ice and drift, as a great part of the material passing over the 500-ft. crest would strike on a width of about 30 ft. The water will be clear, however, and little damage is likely to result. At low discharges, a hydraulic jump will be formed in the conduit where the water moving at high velocity at the outside of the bend, is slowed down by the friction of the horizontal section of the tunnel. As the position of this jump will probably change with the discharge, and only clear water is to be expected, it is not likely to do appreciable damage.

Considerable value would have been added to the paper if the author had given the reasons for the selection of the amount to be used as the maximum head on the spillway, as this enters so vitally into the design. The low head necessitates a wide crest; but there is no apparent reason why a greater head could not have been used, making possible a smaller diameter, fewer gates, and less rock excavation. By using a well shaped ogee spillway, with a head of about 9.5 ft., instead of 7 ft., the length of crest could have been decreased to about half that used, and the danger of drift catching on the crest greatly lessened. This would have reduced the quantity of rock excavation in the approach channels, and as a concrete wall was required under part of the spillway lip in the form adopted, probably no great increase in the quantity of masonry would have been necessary. Such a spillway would have kept the water in contact with the spillway face, and have prevented a vacuum beneath the jet. Although it might not have been the cheapest form for this location, it would be well worth investigating in others.

Could some other form of shaft spillway have been used to greater advantage? It would be interesting to know whether any other shapes were investigated. At first thought, it would seem that a semi-circular, or a fan-shaped, design might decrease the cost, but a closer inspection shows that this is not necessarily so. With certain topographic conditions, considerable saving might result if these shapes could be used. They would have to be carefully studied experimentally, however, to guard against unexpected disadvantages from the hydraulic standpoint.

The writer believes that such experimental investigations generally pay big dividends, and should be used more frequently than they are. The initial expense looks large, and it is difficult to induce a client, who, without question, will invest large sums in an enterprise with only 10 or 15% return, to invest a small sum in such an investigation, which will almost certainly yield this return, and may yield several hundred or a thousand per cent. Perhaps the most striking example of the solution of a difficult hydraulic problem by experiments on models was the development of the hydraulic jump pool for the outlets of the retarding basins of the Miami Conservancy District.* In

* "The Hydraulic Jump as a Means of Dissipating Energy," by Ross M. Riegel, M. Am. Soc. C. E., and John C. Beebe, Assoc. M. Am. Soc. C. E., Miami Conservancy District, Technical Reports, Pt. III.

this case not only the shape used, but the method itself, was determined as the result of the experimental work. Although the expense was considerable, it was amply justified by the saving in cost of construction.

Little fault can be found with the author's analysis of the hydraulics of the form of spillway used. The principal objection is to the neglect of the acceleration which takes place in a stream of water as it approaches a free drop, such as that which occurs at the beginning of the parabolic curved section. The hydraulics of this acceleration has been calculated by S. M. Woodward, M. Am. Soc. C. E.,* for a level bottom above the free discharge. Under these conditions, it was shown that the stream approaching the drop must flow at greater than the critical velocity, and this no doubt would also be true for the case of the spillway. If the up-stream corner of a flat-crested weir is rounded with a sufficiently large radius, the critical velocity would be reached at the up-stream edge of the flat part of the crest, and the velocities down stream from this would be greater than the critical value, although for a very wide crest, friction might cause the point of critical flow to be farther down stream. If the radius of curvature of the up-stream corner is small, the point of critical flow would occur near the lower end of what the author calls the "normal acceleration section", and below this, subject to the friction effect already noted, the flow would be at a velocity greater than the critical, and, therefore, greater than that assumed by the author in computing the parabolic curve. Although the piezometric readings do not indicate a partial vacuum beneath the upper part of the nappe on the model it should be remembered that the flow for the conditions assumed in the design was only 3 in., and surface tension and frictional effects undoubtedly caused greater relative losses than would occur in a full-sized spillway.

Practically, however, it is probable that the parabolic curve is unnecessary, and that a somewhat sharper curve might be used with resulting economy. The objection to the vacuum beneath the nappe on an overflow dam is that it tends to overturn the dam, but this is not likely to be true of a shaft spillway, as the mass and arch action of the weir section would be ample to resist it. With gates on such a spillway, some of which were open, the nappe might tend to jump clear of the masonry surface, and possibly induce some vibration, but this would not be objectionable, as the spillway could hardly fail by sliding.

Summarizing, it is the writer's opinion that the shaft spillway is applicable only to large discharges; that the broad crest should be avoided by using larger maximum heads, reducing the tendency to clogging, and probably the cost; that the disadvantage of having a limiting discharge is of no importance if sufficiently conservative values of run-off are assumed; and that the entraining of air is not likely materially to reduce the discharge capacity.

I. GUTMANN,† ASSOC. M. AM. SOC. C. E. (by letter).—The writer wishes to adduce some European precedents in shaft spillway design.

* "Hydraulics of the Miami Flood Control Project," Miami Conservancy District, Technical Reports, Pt. VII, p. 136.

† New York, N. Y.

The Krauserbauden masonry dam, in what is now Czechoslovakia, is provided with an auxiliary circular shaft spillway discharging into a tunnel which, in turn, empties into a pool formed by a weir built about 100 yd. down stream from the toe of the dam. The spillway shaft is about 16½ ft. in diameter and has a capacity of 3 500 cu. ft. per sec. for a weir head of 3.5 ft. Unlike the Davis Bridge Spillway, it has a distinct spill-weir rising 2 to 3 ft. above the ground. The spill-weir has a rounded crest and supports a vertical cylindrical screen about 10 ft. high. A circular footbridge surmounts the screen.

The masonry dam at Königreichwalde, also in what is now Czechoslovakia, is provided with two circular shaft spillways, one at either abutment, with tunnels leading into a water-cushion at the toe of the dam. The two spillways are not equal in capacity, one carrying about 4 000 cu. ft. per sec. and the other about 2 500 cu. ft. per sec. In their design, they are quite similar to the Krauserbauden shaft spillway.

Both the Krauserbauden and the Königreichwalde dams were built shortly before the beginning of the World War, by a Commission of the Kingdom of Bohemia ("Landeskommission für Flussregulierungen im Königreiche Böhmen"), appointed to form detention reservoirs for flood protection.

A multiple spillway, consisting of three shafts, 8.4 ft. in diameter and more than 60 ft. in depth, was designed by Professor L. V. Rossi, of Padua, for the Bassano Power Canal in Northern Italy.* The shafts are in a row, parallel to the axis of the canal; their upper ends are round-crested circular weirs rising 1.73 ft. above the ground, each weir having a perimeter of 79 ft., so that with a head of 2.46 ft., the capacity is expected to reach about 1 138 cu. ft. per sec. The three weirs are separated by only 1 m. in the clear from wall to wall. The canal and shafts are excavated in rock and lined. Ten grizzlies, 2.5 m. apart, are built in the vertical section of each shaft, in order to break up the force of the falling water.

The San Dalmazzo masonry dam in Northern Italy is provided with a unique spillway of ingenious design.* It is a multiple rectangular shaft spillway discharging into an open waste channel directly beneath. The dam, which is 190 ft. high, is situated in a narrow canyon 4 500 ft. above sea level, and controls a water-shed of 19.4 sq. miles, with a probable flood run-off of about 4 000 cu. ft. per sec., or 206 cu. ft. per sec. per sq. mile. A spillway length of 656 ft. was desired, but only a small, rocky plateau, 130 by 60 ft., was available as a site. The problem was solved by Fraccaroli and Piccinini, engineers of Societa Negri, in the following original manner: The plateau was excavated to a depth of from 6.6 to 11.5 ft. and a width of from 16 to 23 ft., to serve as the inlet to the waste-way. This inlet was bridged by a series of eight parallel concrete flumes resting partly on rock and partly on the walls of the inlet, with clearances of 6.9 ft. between flumes. The waste water enters the flumes from both ends and spills over their sides and through the clearances into the waste-way beneath. The first two flumes, beginning up stream, are

* For details of the spillways of the Bassano Power Canal and the San Dalmazzo Dam see *Giornale del Genio Civile*, July 31, 1917, pp. 341–343, and Pl. IX.

24 by 6.5 by 3.3 ft. deep; the remaining six are about 41 by 7 by 3.3 ft. deep, and the total structure is about 125 ft. long by 43 ft. wide. The total water-spilling perimeter is about 718 ft., that is, greatly in excess of the 656 ft. originally desired. In the winter of 1916-17, this spillway operated successfully with a head of about 1.7 ft. The stream carried ice floes from 1 to 1½ ft. thick at the time.

The author may be underestimating the reduction in capacity resulting from entrained air. Observations by the U. S. Reclamation Service on concrete chutes of the Boise Project in Idaho showed that at velocities of 15 to 30 ft. per sec., and depths of 3 to 4 in., the stream may entrain enough air to swell its original volume by from 30 to 50 per cent. The volume of air so absorbed seems to vary directly with the velocity and inversely with the depth of the stream. At least, the relation of volume of air entrained to velocity appears to vary asymptotically, the volume of absorbed air becoming noticeable in comparatively long runs at velocities of, say, 10 ft. per sec. This condition, therefore, can hardly be foretold from experiments on small-scale models in which only low velocities are developed.

This brings out a limitation of the method of testing by models which may be eminently efficient in the study of straight-line and of curvilinear variations of a non-asymptotic nature, but will seldom prove applicable to the study of asymptotic variation.

Ford Kurtz,* M. Am. Soc. C. E. (by letter).—As indicated by its title, the paper treats exclusively of the hydraulic features of the Davis Bridge Spillway. However, since the adequacy of the spillway provisions for any dam is always a matter of great moment, particularly when dealing with one of the highest earth dams yet constructed, and because the matter has been brought up in several of the discussions, a few remarks on the subject are presented herewith.

The general features of the design of any engineering structure are determined in varying proportions by two major considerations, namely, economy and insurance. The first is purely financial in its scope; the second is both financial and social—financial in so far as it bears on the prevention of damage to or the destruction of the structure by the forces imposed on it, and social in its relation to the safeguarding of human life. In all points decided by economy it is comparatively easy to justify additional capital expenditure that will produce additional income. On the other hand, it is more difficult to decide how much should be expended for insurance, for here enters the arbitrary factor of safety—so often facetiously termed the "factor of ignorance"—and the elusive theory of probabilities. Perhaps engineering designs differ more in this than in any other respect and certainly herein lies the principal difference between the conservative and the radical in design.

A steel penstock supplying water to an hydraulic turbine furnishes a typical example of a structure the design of which involves both the considerations mentioned. The larger the diameter the greater is the cost, but at the same time the smaller is the resultant loss of head, the greater the resultant power

* Care, J. G. White Eng. Corporation, New York, N. Y.

developed by the turbine, and the better the speed regulation under commercial load changes. On the other hand, the thickness of metal used in the pipe walls can have only a minor and indirect influence on the loss of head and the speed regulation. The problem involved amounts to, "How much shall be spent for insurance?" Shall the factor of safety be 4 under the maximum static head, or under the condition of increased pressure which will obtain when the turbine gates are rapidly closed by the governor? Shall the factor of safety be applied to the average ultimate strength of the metal or to the minimum? In general, the tendency is to decrease factors of safety with the increased certainty as to the maximum forces that can be imposed on a structure, for in the last analysis the factor of safety is only the means of providing for unforeseen and incomputable stresses—of allowing for "scientific ignorance".

In contrast to a penstock, the spillway feature of any dam is purely a matter of insurance. Increasing the spillway capacity does not add a cent to the income from the development; the spillway is simply a safeguard against the destruction of the dam by flood waters in excess of the flow that can be passed through the turbines. The spillway is strictly a "nuisance" problem, but, nevertheless, one of the most important confronting the designer of dams. Foundation problems may be complex, but the ground is there, ready to yield its secrets, and awaiting only the attack of the pick and the core drill; but who can project his vision into the future and predict the magnitude of floods in the years to come? Here, indeed, is a problem worthy of the most skilled investigator.

For this reason the Engineering Profession is deeply indebted to those members of the Society, who nearly ten years ago set forth the application of the mathematical theory of probabilities to this question of the frequency of floods of different magnitudes. They showed the way out of a maze of erroneous conceptions by demonstrating that there is no such thing as a maximum flood. Engineering literature has also been greatly enriched in recent years by the studies of great storms and flood frequencies, such as those made by the engineers of the Miami Conservancy District. The study of spillway capacities along these new lines, marks a distinct advance away from "scientific ignorance" and nearer a clear conception of the safety of dams against overtopping by floods.

In the case of the Davis Bridge Spillway, records of stream flow at several points on the river have been kept by the U. S. Geological Survey for a number of years, and private records have also been kept by the New England Power Company at its various plants since their completion. The length of the record was about 13.5 years at the time the studies for spillway capacity were made. The flood records at all the stations were compiled and analyzed and from them the highest twenty-seven flood peaks at the Davis Bridge Dam site during that period were estimated. These peaks when plotted on the probability basis showed that a flood peak of 27 000 sec-ft. might be expected once in 333 years, and this rate was chosen as the nominal capacity of the spillway. It is to be noted, however, that one-half of the twenty-seven flood peaks

used occurred prior to the completion of the Somerset Dam and are higher than would now be produced by the same conditions, as the Somerset Reservoir is so large that not a single drop of water has ever flowed over the spillway since its completion. Consequently, the probable frequency of a 27 000 sec-ft. flood is undoubtedly less than once in 333 years.

However, this probability does not present an adequate idea of the safety or insurance factor, because no account has yet been taken of the effect of storage above spillway level in smoothing out the sharp flood peaks, nor of the likelihood of the reservoir being less than full when the great flood occurs. As for the storage effect, even if flash-boards 6 ft. high are in place with the water level at their tops when this tremendous flood comes and even if the boards are not tripped until the water is flowing 2 ft. deep over them, the storage above this level is so large that a flood peak of 34 000 sec-ft. on a day when the 24-hour average flow is 28 000 sec-ft. would only raise the water level 2 ft. higher, that is, to a point 10 ft. above spillway crest level. This condition represents a flood probability of once in 5 000 years. The ratio of peak flow to daily average for use in the storage computations is that indicated by the Fuller formula.* Combining the probabilities of these separate occurrences—the 5 000-year flood, its occurrence at a time of full reservoir, and the failure of the operators to trip the flash-boards earlier—so as to obtain the probability of the simultaneous occurrence of all these events, would give proper measure of the safety of the dam. Certainly, this probable frequency would not be greater than once in 25 000 years. There is, however, still another factor to evaluate. The tests on the model indicated a capacity of the full sized spillway in excess of 32 000 sec-ft. with an increase of only 1 ft. above the design head of 8 ft. Furthermore, these results were obtained on a model the surfaces of which were given a roughness corresponding to a value of $n = 0.014$ in Kutter's formula, which represents very rough concrete work. Thus, it is seen that many elements of safety exist beyond the merely formal spillway capacity of 27 000 sec-ft., corresponding to a 333-year flood.

Viewed from still another angle, the nominal capacity corresponds to a run-off of 6¾ in. in 24 hours over the entire tributary area of 154 sq. miles. In a depth of 3 ft. above flood level a run-off of ¾ in. in 24 hours may be stored, giving a combined flood capacity equivalent to a run-off of 7½ in. in 24 hours from the entire water-shed. As compared with this, the greatest recorded 24-hour precipitation in this region,† amounted to about 8 in.

Again, studies made by W. P. Creager, M. Am. Soc. C. E., to supplement the flood curves presented in his book on "Masonry Dam Design", indicate that the worst flood-producing streams of a drainage area of 154 sq. miles would have a 10 000-year flood peak of 47 000 sec-ft. The storage of 3 ft. above flood level would carry the excess of this peak over and above the nominal spillway capacity of 27 000 sec-ft. for a period of 3.5 hours, or would carry such a peak building up from 27 000 sec-ft. to 47 000 sec-ft., and dropping down again in 7 hours. At the peak of such a flood the run-off rate would be about ½ in.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 567.

† Miami Conservancy District, Technical Reports, Pt. 5, "Storm Rainfall of Eastern United States", p. 138.

per hour from the entire water-shed—truly a tremendous rate for such a large area.

As the selection of a shaft spillway for the Davis Bridge Dam has been questioned in some of the discussions, the writer desires to list the other types considered and the reasons for their rejection.

Open Channel Spillways.—

(a).—Diversion at a Low Saddle into Another Water-Shed.—The topography did not lend itself to this scheme.

(b).—Straight Overflow or Weir Spillway in Line with Axis of Earth Dam.—The river occupies a pre-glacial gorge filled with drift. Rock is available on one side of the gorge only and over an area insufficient to permit the construction of 500 ft. of masonry spillway and of the necessary abutment for an earth dam, except at prohibitive cost.

(c).—Re-entrant Overflow or Weir Spillway with Discharge Channel.—The hydraulics of the discharge channel are uncertain, requiring the adoption of ample dimensions to insure the capacity desired. The resulting deep trench or slot excavation in rock would have been excessively costly.

Closed Shaft Spillway.—

(a).—Siphon Inlet Type.—The form considered was circular in plan and symmetrical. It was rejected because of (1) complicated structural problems involved in the design and construction of the cover or roof; (2) the deep excavation for the approach channels to the submerged openings; (3) the danger of formation of ice over the submerged openings if the water level in the reservoir should be near the level of the openings; and (4) the danger of clogging the channels near the throat due to the convergence necessitated by the circular form.

(b).—Open Bellmouth, or Funnel, Inlet Type.—Four forms were considered, namely:

(*1*).—*Clear Crest.*—This type was not adopted as the addition of piers improved flow conditions as well as permitted the use of flash-boards.

(*2*).—*Piers with Flash-Boards.*—This was the form adopted.

(*3*).—*Piers with Taintor Gates.*—This type of gate was not considered desirable for the spillway of an earth dam.

(*4*).—*Rectangular Slot with Bottom Sloping Toward Shaft.*—Tests were made on a model of this type but the interpretation of the results was too uncertain to justify basing a design on them.

In view of the data submitted by Mr. Gourley on the Taf Fechan Spillway designed in 1911, the writer withdraws his statement that the Davis Bridge Shaft Spillway with its bellmouth, or open funnel, entrance is a new type of shaft spillway. Mr. Gutmann has presented some interesting data on unusual entrances to shaft spillways, but none of the examples cited appears to be of the bellmouth entrance type, nor does the Calaveras Shaft Spillway described by Mr. Hazen come in this class.

The paramount importance of a well designed bellmouth, or funnel, entrance in eliminating as far as possible the entraining of air by the flowing water

does not seem to have occurred to Mr. Muckleston. The inlet of a Taylor air compressor is especially designed for the purpose of air entrainment and, as Mr. Muckleston testifies, "it works." On the other hand the Davis Bridge Spillway entrance is especially designed so that, at design capacity, it will minimize the entrainment of air and in the case of the model the writer can testify that, in the words of Mr. Muckleston, "it works." For flows below the design capacity both the model and the full-sized Davis Bridge Spillway do carry air through the shaft and tunnel as evidenced by the strong draft at the outlet of the tunnel in both cases. This can certainly do no harm; it does not reduce the design capacity and Mr. Hazen suggests that entrained air *per se* is beneficial. Mr. Gutmann's discussion refers to the entrainment of air in the case of long open-channel flow on steep slopes. It seems to the writer that, in designing this shaft and tunnel, it is much more far fetched to apply deductions from full-sized open-channel flow than from hydraulically homologous models, as criticized by Mr. Gutmann. Probably every engineer who has had experience with testing by models realizes the limitations of the method, but agrees that it is certainly much better in general than the drawing of deductions from full-sized tests on entirely different structures.

Mr. Muckleston's statement that flow at the so-called critical depth (that is, a depth equal to twice the velocity head of the flowing water) can obtain only at the down-stream edge of a broad-crested, level, flat-topped weir is so erroneous and misleading that the writer desires to state categorically that this is the one place where it is absolutely impossible for flow at the so-called critical depth to exist. So simple is the theory of critical depth flow it is difficult to understand how any one could fail to remember that the fundamental assumptions are those of a uniform velocity throughout the cross-section of the flowing water and, as a corollary, of the existence of a pressure head at any point in the cross-section equal to the depth of that point below the upper or free water surface. On the other hand, at the down-stream edge of any unsubmerged weir, the pressure at all points throughout the cross-section of the flowing water is practically atmospheric and the velocities are widely varying, being smallest at the top and largest at the bottom of the cross-section—conditions subversive of the assumptions of the theory of critical depth flow. If Mr. Muckleston desires experimental confirmation of his error and the correctness of the writer's analysis for depth of flow at the down-stream edge of a flat-crested weir, he is referred to measurements of depths at that point, made by the U. S. Board of Engineers on Deep Waterways.* Mr. Muckleston's remarkable pronouncement to the effect that the non-uniformity of the velocity in the cross-section of a flowing stream accounts for the disagreement between the various values of C in weir formulas and the value of 3.087 obtained by theoretical treatment, and for most of the discrepancies between prophecy and performance, will certainly be received by most engineers with as much incredulity as it has been by the writer. Perhaps no theory should ever be derided, but until Mr. Muckleston presents very substantial arguments to back his theory the writer will adhere to his belief that eddy losses, due to contraction conditions at the up-stream edge of weirs, explains the major

* *Water Supply Paper No. 200*, U. S. Geological Survey, p. 111.

portion of the variations to account for which Mr. Muckleston advances his new theory.

Mr. Gourley has not furnished any information as to the height of the Taf Fechan Spillway Shaft, the length of the tunnel, and the design capacity and depth on the crest, so that it has not been possible to analyze the design in accordance with the methods of the paper. With these data lacking, the only question is that mentioned by Mr. Lane in his discussion, namely, the sufficiency of the radius of the bend connecting the shaft and tunnel for passing long logs without wedging. In assuming that the flow over any individual weir or over homologous weirs conforms to the equation, $Q = C\ L\ H^m$, in which, C and m are invariable constants, Mr. Gourley is faced with a vast amount of experimental data which will rise up to plague him unless he completely rejects them. One example from the data of the 1877 experiments of Fteley and Stearns on sharp-crested weirs will suffice. The experiments chosen have practically the same heads as the two selected by Mr. Gourley from the writer's tests, and the length of weir is 5 ft. The points selected lie very close to the mean curve for all the experiments:

Head, in feet.	Flow, in second-feet.
0.0746	0.3652
0.1627	1.1367

Analyzing these two sets of values by Mr. Gourley's method gives the following formula for a sharp-crested weir:

$$Q = 3.20\ L\ H^{1.456}$$

Would any one give serious consideration to such a formula for heads up to 8 ft. on a large sharp-crested weir? Assuredly not, and yet Mr. Gourley throws grave doubts on the capacity of the Davis Bridge Spillway on the basis of an exactly similar method of analyzing the writer's experimental data for the model. It furnishes an example of the dangers of making a fetish of the idea that all hydraulic experimental data when plotted on logarithmic paper must lie on a straight line.

No engineer will contradict Mr. Jakobsen's remarks on the advantages of the ordinary weir spillway provided only there is ample capacity in the discharge channel leading away from it. Undoubtedly, it is an ideal type for earth dams, but like many ideals, it cannot always be realized. Nature was not favorable to the realization of the ideal at Davis Bridge, for, as already explained, the difficulties in the design, and the cost of construction of the discharge channel leading away from an ordinary weir spillway were the very things that led to the adoption of the shaft type. Mr. Jakobsen states that "great accuracy in determining the spillway capacity is not required, because the maximum flood is not known with any degree of certainty" and that "for earth dams, an excessive spillway capacity should be provided * * *." It seems to the writer that the desired spillway capacity must be defined in more accurate terms than "excessive." Having done so it certainly is good design to have the knee of the curve in Fig. 25 lie below the crest of the dam if the capacity at the knee equals that desired.

Both Mr. Muckleston and Mr. Jakobsen question the hydraulics of the cylindrical shaft and the tunnel, particularly in the application of Kutter's

formula to such high velocities and in the matter of loss of head in the bend and transition. It is true that there are no data on the loss of head at velocities as high as 68 ft. per sec., but there are data on velocities as high as 20 ft. per sec. in concrete lined tunnels as large as 18 ft. in diameter, and these data show no tendency for Kutter's n to increase with the size of pipes and the velocity—rather the opposite, if anything. S. M. Woodward, M. Am. Soc. C. E., who was in charge of the hydraulic design of the outlet conduits for the retaining dams of the Miami Flood Control Project, has very concisely set forth his opinion on this matter in a letter to the writer, as follows:

"Since there appeared to be no definite change in the value of n for velocity ranging from a few feet per second up to 20 ft. per sec., it seemed safe that there would probably not be much change in n for high velocities up to 50 ft. per sec."

It cannot be denied that present knowledge of the loss of head in bends is meager and confined to pipes of small diameter; but must engineers for that reason abandon all projects involving pipes or tunnels having curves?

Mr. Kennison mentions the positive limitation of the capacity of the Davis Bridge Spillway, but it must also be borne in mind that the ordinary spillway if used in this case would have been of the reentrant type with a discharge channel and that its capacity would have been limited by this discharge channel just as much as the shaft spillway is limited by its discharge conduit. The uncertainties in the hydraulic treatment of the open discharge channel are much greater than in the case of a closed discharge conduit, so that more liberal assumptions must be made in the former case, with the result that for a given capacity the closed conduit or tunnel was cheaper than the open discharge channel. His further suggestion of a straight spillway along the contour of the hill in place of the symmetrical bellmouth or open funnel was fully considered and experiments were made on a model of that type. It was not adopted because of the difficulties of mathematical treatment and the consequent uncertainties in drawing conclusions from the results of the tests on a model. The excavation for the spiral approach channels to the symmetrical type was much cheaper per unit than for the deep discharge notch or channel that would have been required for the ordinary weir type of spillway. Mr. Kennison directly states, and Mr. Hazen, by his strictures on the writer's mathematical treatment, intimates, that the exact shape of the crest to pass the given discharge at low head is of little consequence. The writer cannot agree with this idea for three reasons: The correct shape of crest (1) gives the minimum of loss of head; (2) involves the minimum of excavation; and (3) reduces as much as possible the entrainment of air.

The difficulties in the interpretation of experiments on models as pointed out by Mr. Parsons are fully recognized by all, but if the resulting deduction that "results obtained by observation and measurement should not be extended beyond the limits of experimentation with the model" is accepted, there will be no testing except on full-sized models. Surely, Mr. Parsons does not advocate such a radical step. He expresses a belief that air will be entrained and may set up a vibratory action. Fortunately, the rock through which the shaft and tunnel were driven is sound and massive, and the lining is well placed, so that

no fears are entertained as to the results of any vibratory action. A boom will be provided to prevent logs from passing over the spillway. As surmised by Mr. Parsons, construction costs and conditions precluded the use of a compound curve or a longer radius curve at the foot of the shaft.

Mr. Dunham's questions as to the structural design of the spillway are fully answered in an article on the Davis Bridge development, published elsewhere.* The reason for his reference to low temperatures in shafts is not clear, unless he fears the formation of ice in the tunnel and shaft with the resultant possible blocking at time of flood. No difficulty is anticipated in this respect, but if it does arise, obviously steps must be taken to prevent it.

Mr. Halmos attempts to describe what he considers a better method of accelerating the water into a shaft, but confines himself to such vague, general terms that the writer does not feel called on to discuss its merit or lack thereof. Mr. Halmos intimates that he knows the explanation of the existence of the partial vacuum which developed in the experiments on the Davis Bridge models; if so, it is deeply regretted that he did not give it instead of expressing amazement that others had not already discovered the reason. He certainly had no authority for his statement that this partial vacuum resulted in sucking air and causing tremor in the model, for neither of these things happened.

Mr. Comstock has approached the subject of the sufficiency of the spillway from the logical standpoint of the magnitude of floods to be expected, in striking contrast to those who have criticized the design because the spillway reaches its practical maximum capacity at a head less than the total freeboard of the earth dam above the spillway crest. His views on flash-boards must inevitably lead him to condemn all types of crest gates—a decidedly ultra-conservative stand in the light of present-day practice. The subject is, however, not closely enough related to the paper to justify a general discussion at this time.

Professor Barrows' investigations form a valuable contribution to the subject of shaft spillways in general and the Davis Bridge Spillway in particular. It is a source of satisfaction that Professor Barrows' results both as to the sufficiency of the spillway provision and its capacity agree so closely with the previous determinations. It is perhaps in order to explain why readings on piezometers in the model of the tunnel were not taken during the Davis Bridge experiments. In the first place it was realized that the tunnel would not be full throughout its length, except for flows approaching its maximum capacity, and, therefore, piezometer readings would be of no particular value. In the second place, the experiments† of E. W. Schoder, Assoc. M. Am. Soc. C. E., indicated that the disturbance in pressure due to a pipe bend exists for a considerable distance down stream from the end of the bend—perhaps, as far as 70 to 100 diameters. Such a condition would render piezometer readings a short distance down stream from a bend of doubtful value, especially in studies of the excess loss of head caused by the bend.

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* *Engineering News-Record*, January 24, 1924.

† "Curve Resistance in Water Pipes", *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 67.

The writer believes there was some error in Professor Barrows' Series 5 under experiments on the open crest type (Table 11). This series gives a point which, when plotted on Fig. 28, falls in line with points for the crest with flash-board piers. This does not seem reasonable, especially in view of the fact that Series 6 and 7 give points which fall well above the average curve drawn through the test points for the crest with piers.

Mr. Scheidenhelm states that there are few, if any, precedents in the way of structures intended to endure velocities as high as the design velocity in the Davis Bridge Spillway shaft and tunnel. Perhaps the most notable structures of this type are the outlet conduits of the retaining dams of the Miami Conservancy District for which the design velocities in concrete conduits range from 40 to 50 ft. per sec. As pointed out by him, the importance of a smooth surface on the concrete lining in preventing local erosion cannot be over-emphasized and every endeavor was made at Davis Bridge to secure as smooth a surface as practicable. Fortunately, the rock is very firm and dense so that there is no probability of serious damage even if some erosion of the lining should occur and, of course, there will be ample time and good facilities for making necessary repairs after any flood that might cause such erosion.

Mr. Lane has presented an able and constructive discussion. The drift problem was fully appreciated. As already mentioned, measures will be taken to prevent long pieces from entering the spillway. The adoption of a low head for passing the design flood was on a purely economic basis. The additional storage thus obtained justified the additional expenditure on the spillway. Many other types of spillway entrance were considered, one of which was tested by means of a model, but the circular, symmetrical type was adopted because of its simpler hydraulics and the belief that model results for such a type could be interpreted with greater certainty than for non-symmetrical types.

The writer cannot agree with Mr. Lane when he states that acceleration of the water as it approaches the free drop was neglected. A review of the writer's analysis will show that at the beginning of the free drop every particle in the jet is assumed to have a velocity corresponding to its vertical distance below the water surface in the pond, after making due deduction for losses over the flat crest. Can the particle have any greater velocity than this unless a partial vacuum exists in the lower portion of the flowing stream (a rather unusual assumption and one not borne out by the experiments)? If not, how has the acceleration been neglected? True, there has been no attempt to determine the shape of the draw-down curve as the water approaches this edge, but that is not necessary, unless one attempts to determine the angle of the mean thread of the jet to the horizontal, instead of assuming it to be horizontal, as was done by the writer. The advisability of a sharper crest curve than that adopted at Davis Bridge is open to grave question.

In conclusion, the writer thanks all those who have contributed discussions and thus added to the store of ideas and information on the subject of shaft spillways.

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Paper No. 1552

SANITATION—ITS RELATION TO HEALTH AND LIFE*

By George C. Whipple,† M. Am. Soc. C. E.

With Discussion by Russell Suter, M. Am. Soc. C. E.

In an able address‡ before the Sanitary Engineering Division of the Society on January 16, 1923, Harrison P. Eddy, M. Am. Soc. C. E., described the present status of sanitary engineering and outlined its scopes and activities. The speaker wishes to present an even more fundamental aspect of sanitation, namely, its relation to health and life.

The subject cannot be discussed logically without first considering some of the words commonly used in connection with it. What is meant by health? This word comes from the old Anglo-Saxon word, "haelth". There are several allied words, such as "hale", "whole", "wholesome", and "holy". This idea of wholeness seems to be fundamentally involved. Health applies to the whole body. The common expression, "all right", gives the proper idea. Health is bodily "all rightness".

The word, health, is used in two ways: Absolute and relative. It may refer to a state of physical and mental wholeness to the exclusion of some lower state, that is, health may mean normal health or good health; or it may represent a designated relation to such a normal state, as when one says excellent health, bad health, or some other variety of health. Giving the word its full meaning, it is possible to analyze it, or, at least, to state its concepts.

Health implies organic soundness. It is stated in the Bible that "they that are whole need not a physician * * *." It implies a condition in which all the organs function naturally and the body as a whole operates effectively.

* Presented before the Sanitary Engineering Division at New York, N. Y., January 15, 1924.

† Cons. Engr.; Prof. of San. Eng., Harvard Univ., Cambridge, Mass. Professor Whipple died on November 27, 1924.

‡ *Proceedings*, Am. Soc. C. E., January, 1924, Papers and Discussions, p. 3.

It implies a condition in which the body responds adequately to its environment and in which there exists a sense of comfort, or physical contentment. It implies also mental soundness and mental activity. In other words, full natural health has four elements: A sound mind, a sound body, natural functioning, and fitting responses to outside influences during a considerable period of time.

It is not surprising that some people should pick out one of these concepts and emphasize it to the neglect of the others, thus giving to the word, health, a limited or circumscribed meaning. Some limit the word to the soundness of the physical body, some exclude the senses, some overlook the nervous system and mental state. To many physicians, and to some health officers, health means merely the absence of disease. The increasing use of vital statistics, based on deaths and cases of sickness as measures of the healthiness of a community, is tending to spread this unfortunate partial use of the word. The positive elements of health are indeed more difficult to measure than the negative departures from good health; but, nevertheless, there is a positive side which deserves consideration. There is a condition in which vitality is being stored up, just as there is a condition in which it is being exhausted. Some day, perhaps, physiologists and psychologists will find better ways to measure and describe health. As an old student of vital statistics, the speaker will state that death rates are an inadequate measure of the health of a community. Statistics of sickness are available only for certain major diseases, whereas the total incapacitating influence of the more numerous minor ailments and disorders is probably greater than that of the diseases reported. One of the most difficult problems which vital statisticians have before them is that of finding and applying an adequate mathematical index of the health of a community. To define what is meant by normal health is difficult, because the normal is not the same at all times or in all places. Biometrics is a promising science; thus far, however, it has given little practical help to health officials.

Environmental Complements to Health

One cannot get far in a discussion of health without taking account of environment; in fact, it is difficult to conceive of man apart from his environment. Normal health connotes a normal environment. Health is dependent in great measure on inborn, constitutional, hereditary factors. Length of life depends more on these intrinsic forces than on anything else. The environmental factors, however, are of especial interest to sanitary engineers. They are more readily controlled than the personal and hereditary factors. Some environmental factors injure health; others promote it. Omitting the constitutional phases of the subject, three classes of injurious environmental factors may be distinguished:

1.—There are damaging factors, which include infections, poisonings, and accidents. The biological, chemical, and physical injuries inflicted by them tend to drag health downward. Whatever benefits they offer are only by way of warding off worse injuries.

2.—There are what may be called functional factors, which have to do with breathing, eating and drinking, excretion, sleep and exercise, body temperature, and growth. Depending on their character, they affect the body either for good or for ill; they may injure health or promote it; they are either negative or positive. Good food may promote health; poor food may injure health. Good food, pure water, fresh air, proper lighting, and exercise tend to create a store of vitality; poor food, impure water, foul air, inadequate lighting, and fatigue tend to deplete the health reserves. These matters to a large extent come under the head of hygiene.

3.—There are also sensatory factors, that is, environmental influences which affect the perceptions and, through them, the bodily functions. These also are positive or negative. Bad smells and tastes are injurious; whereas agreeable odors and tastes are beneficial. Excessive noise is injurious; music is beneficial. Sights of ugliness and cruelty are injurious; beauty is a great uplifting force. In these matters, the mental and the physiological are so intertwined that they cannot be kept apart by definition, and it is hardly worth while to try to do so, if health is given the broad interpretation understood by the man on the street.

Of the three kinds of environmental influences, those which damage the health by infection, poisoning, or physical injury are of major importance. Exclusive of infections, the effect of the quality of air, water, and food on human health is relatively slight. The sensatory or perceptional factors are of even less importance. Nevertheless, these minor factors which injure health indirectly should be recognized and their influence not denied because they cannot be evaluated comparatively. Sometimes their effect is great.

Fig. 1 indicates the three classes of factors and their relative importance. The lines are not drawn to any scale, because the required data do not exist.

Comfort, Vital Abundance, Happiness

Although some would limit health to the absence of disease, others would have it include comfort, fullness of life, efficiency, and happiness. Some of these higher expressions characterize health, but do not define it. Health leads to happiness, but health is not happiness.

In his recent book on "Personal Hygiene Applied", Dr. J. F. Williams, of Columbia University, has defined health as "the quality of life that renders the individual fit to live most and to serve best." Here, emphasis is placed on the motive for health rather than on health itself. The word, comfort, seems to be so closely related to health as to be inseparable from it. Both in popular writings and in laws, the words, "health and comfort of the people", are found. It is significant that the laws do not say health or comfort. The ideas are conjoined. The senses are closely related both to the physiological and the mental. Through them, one experiences physical comfort and mental contentment, or physical discomfort and mental dissatisfaction.

Furthermore, it must be admitted that just as the body, its functional activities, and its sensations, may affect the mental state, so the mental state may affect the functions of the body. The two are not antagonistic, but com-

plementary, and perfect health results from what Carlisle called their "sweet co-operation."

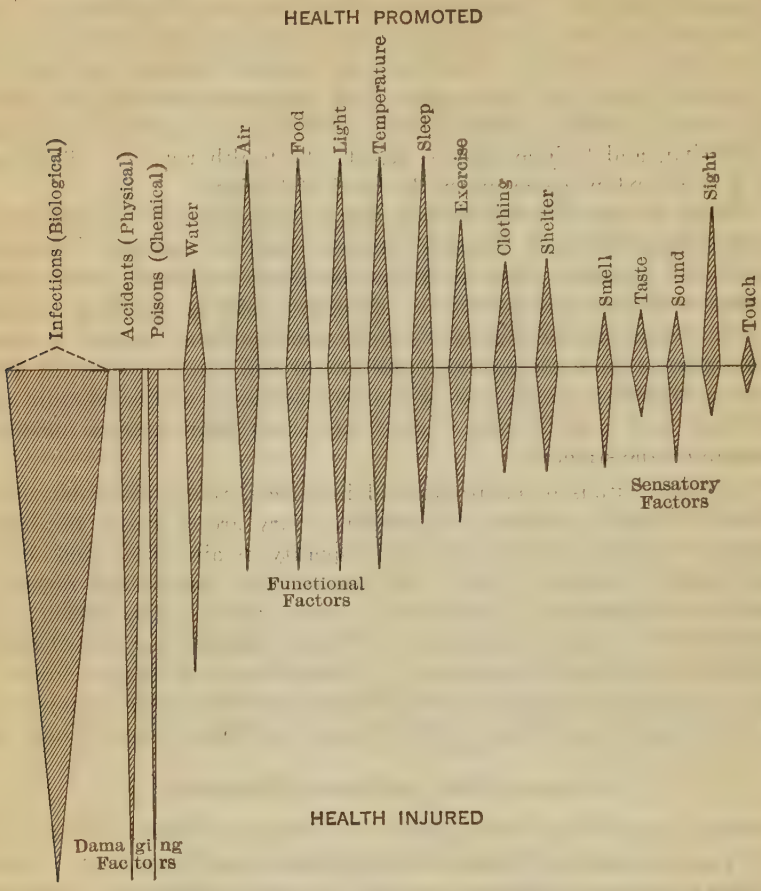

Fig. 1.

Nuisances and Annoyances

When the environmental factors previously mentioned decrease in magnitude and duration, their influence on health may be so small as to be practically negligible. They become mere annoyances or inconveniences. A slight cold, a sliver in the finger, the occasional loss of sleep, a fleeting unpleasant odor may annoy without doing actual injury to health. Obviously, there are quantitative factors and time factors which should be recognized. People differ as to their sensitiveness to environmental factors. A mere annoyance to one person may be actual injury to another.

The word, nuisance, is so largely a legal term that it is best not to use it in any other sense. A nuisance may be a mere annoyance, a discomfort, or it may be an injury to health, safety, or morals. Blackstone said that a "nuisance is anything that worketh hurt, inconvenience, or damage". Modern

legal thought adds the ideas of substantial damage and unreasonable use of property and liberty. A nuisance is a question of fact. As Jeremiah Smith stated, "A nuisance is whatever the Court says is a nuisance", and, of course, the jury must base its verdict on the evidence in the particular case. There are, to be sure, statutory nuisances. There are some things which are considered to be nuisances *in esse*. For example, in some places, the mere emission of black smoke from a chimney is declared by statute to be a nuisance; hence, in a particular case in Court, it is necessary to prove the existence of the smoke, not that the black smoke in question injures the health and safety or annoys people.

Public Health

Public health is usually understood in a general and vague way to mean the collective health of the people in a given community. It is measured negatively and expressed very crudely in terms of birth rates, death rates, sickness rates, and the like, little account being taken of what may be called the minor diseases and departures from health and no account being taken of any of the positive elements of health. Although no adequate definition of public health has been formulated, the term has grown in popularity until it has become almost a fetish. It is replacing the word, health, in unreasonable ways. State Boards of Health are becoming Departments of Public Health. Schools affording training in this branch are called "Schools of Public Health", whereas the term, "Health School", would probably be better. The adjective, "public", limits the word, "health". Health is more inclusive than public health. The International List of the Causes of Death contains the titles of more than two hundred diseases, all, of course, are injurious to health; but of these diseases only about fifty are commonly declared by health authorities to be dangerous to the public health. These are chiefly the more serious of the communicable diseases. Yet, at present, many of the newer activities of health departments are being devoted to what would seem to be individual health rather than public health. Without questioning the desirability of protecting and even promoting individual health, there is a serious question as to how far it is just, wise, and economical to spend public moneys for individual benefits. Some argue that whatever affects the death rate affects the public health. This is not a sound argument, for the death rate itself is a composite of the death rates from many causes, some of which have a public significance and some of which have not. Some argue that the health of every individual has an economic bearing on the whole social community. Some say that, whereas in rural communities individual health remains individual, in a large city the concentration of people causes all health to be public health. It is somewhat anomalous that at the same time that the broad term, health, is being superseded by the narrower term, public health, in governmental administration, the functions of health departments are being extended.

When the environmental factors which affect the public health are considered, the uncertainties are still greater. Infections, such as typhoid fever,

which are broadcasted through polluted water or milk, unquestionably affect the public health; and so do infections which form a continuous chain from person to person, as in the case of common colds, venereal diseases, and infections which are spread by insects. There are infections, however, which are essentially personal and which do not to any great extent spread from person to person. There are some poisonous factors and accident hazards which are likely to affect many people and there are some which can affect only a few. The factors which influence functional activities and the senses affect individuals more often than large groups of people, but a factory which spreads irritating fumes to such an extent that many people are functionally injured through their senses, may fairly be said to injure the public health.

This distinction between injuries to health and injuries to the public health would be of great importance if it were not for the fact that the police power of the State, the common law principle under which Governments act, is not limited to public health, but deals with health. The distinction between benefits to health and benefits to public health is important, because it involves justice in taxation.

Public Nuisances

Although a nuisance is something for the Courts to decide, public nuisances have sometimes been given a statutory basis. In New York State, the Penal Code defines a public nuisance thus:

"A public nuisance is a crime against the order and economy of the State and consists of unlawfully doing an act, or omitting to perform a duty, which act or omission:

"1. Annoys, injures, or endangers the comfort, repose, health or safety of any considerable number of persons; or

"2. Offends public decency; or

"3. Unlawfully interferes with, obstructs, or tends to obstruct, renders dangerous for passage, a lake or a navigable river, bay, stream, canal, or basin, or a stream, creek or other body of water which has been dredged or cleared at public expense, or a public park, square, street or highway; or

"4. In any way renders a considerable number of persons insecure in life or the use of property."

It is interesting to note that the words, safety, health, repose, and comfort, of this statute are grouped together and form a sequence not unlike that of the words, damaging, functional, and sensatory factors, already described, whereas decency is set apart, perhaps, as a moral factor.

In deciding whether something, as, for example, a disagreeable smell, is a public nuisance, it is necessary to consider its magnitude, distance, frequency, and duration of occurrence, incongruity as to time and place, its cause with reference to unreasonable use of property or liberty, and the number, character, and condition of the people affected.

It might be justice for the Court to ignore a strong odor if very rare, or a more frequent odor if mild; to ignore a certain odor in an industrial district, but not in a residential district; to ignore an odor that prevails in the day time, but not at night; to ignore an odor in a sparsely settled region, but

not in a populous region. Consideration should also be given to the effect of the alleged nuisance on persons in feeble health as well as on those in ordinary good health.

Sanitation and Public Sanitation

Sanitation is the art of securing and maintaining a clean environment for the protection and promotion of health and comfort. In common parlance, emphasis is placed on this idea of cleanliness. Things which have to do with safety to life and limb, such as safety appliances against accident or fire, are usually not included under sanitation; but, on the other hand, mosquito suppression is included, even if it is not strictly a matter of cleanliness.

Those parts of sanitation which relate to communities rather than to individual persons and houses, come under the head of Sanitary Engineering, a branch of Civil Engineering. Sanitary engineers now deal with such matters as: (1) Water supply and water purification; (2) plumbing; (3) sewerage, sewage disposal, and river cleaning; (4) disposal of industrial wastes; (5) street cleaning and the collection and disposal of municipal wastes; (6) drainage and mosquito control; and (7) air supply and purification.

In some respects, the sanitation of buildings is a question of architecture rather than of engineering. At any rate, there are in architecture many problems of plumbing, heating, ventilation, lighting, basement drainage, and the orientation of buildings with reference to sunlight, all of which have important relations to health.

It may be worth while to consider how the methods and equipment of the art of sanitation affect health.

Water purification protects against damage by infection and lead poisoning, and against unpleasant tastes and odors. The quality of water is also a functional and sensatory factor.

Sewage treatment, river cleaning, and the disposal of industrial wastes are less important than water purification as far as damage by infection or poison is concerned, but, as sensatory factors, they are very important.

Faulty plumbing is only a slight damaging factor, infections, poisons, and accidents being very infrequent. It may, however, affect the senses of smell, sight, or sound, and may have an influence on the quality of the air, hence on breathing. Good plumbing promotes health by encouraging the use of water.

Inadequate heating and ventilation are chiefly functional factors, involving breathing, temperature control, sleeping, exercise, elimination of water, and growth. They are also sensatory and damaging factors.

Garbage disposal is of very little importance as far as infection is concerned, but has value as a sensatory and functional factor.

Street cleaning involves problems of damage by infection and accident, but the sensatory and functional elements are more important. The same is true of other phases of air sanitation, such as smoke control.

Mosquito control, in some parts of the country, is an important factor in infection; everywhere, it is a sensatory and functional factor.

The branches of sanitation which have to do with housing, namely, heating and ventilation, plumbing, lighting, artificial illumination, and noise preven-

tion, attain their greatest importance as functional factors, although problems of infection, accident, poisons, and the senses may be involved. The more the speaker studies the subject, the more he is impressed with the supreme importance of what is called "housing" on human health. Housing is referred to in its broad aspect, from city planning and the regulation of building on the one hand to housekeeping on the other. The effect of housing is unlike that of the sudden infection of a water supply—spectacular and fearsome—it is slow and insidious and influences health and growth through the senses and the functions. Mathematically, this relation is obscure, illusive, and intermixed with the factors of poverty and ignorance, yet long experience shows that it exists. Rat-proofing is a phase of the housing problem which has a distinct infectional relation.

Industrial sanitation has problems which include many of the foregoing. Accidents, poisonings, and infections predominate, but there are also functional and sensatory factors.

Food sanitation involves important problems of infection, poisoning, and functions; whereas clothing sanitation has a functional bearing.

No attempt has been made to analyze these activities carefully or in detail, but they deserve thought and are often brought in question in the Courts.

An important point is that the object of sanitation is not merely to prevent injury to health, although it is largely that, but also to promote health. Water is filtered to make it clean, not merely to remove the germs of disease. Clean water is health promotive, whereas a chlorinated dirty water is merely health protective. Sunlight and pure air in abundance are emphatically health promotive agencies. These positive benefits of sanitation are too often overlooked. The speaker believes that there is a moral power in cleanliness; and that good sanitation contributes not only to the elimination of disease and positive betterment of health, but to those elements of life which make for better men and women and a higher type of civilization.

There is a tendency on the part of some health officials to belittle sanitation, to emphasize the infections in their personal relations, and to ignore the importance of environmental conditions apart from infections. From the standpoint of disease they are right; from the standpoint of health, the speaker thinks they are wrong. It is probably true that overflowing cesspools, unclean yards and apartments, defective plumbing, dirty cellars, smelling garbage, and smoke, odors, and fumes from factories have a relatively small influence on the death rate as compared with the effect of the major infections which pass from person to person, or when compared with gross abuses of the laws of hygiene. Yet the speaker thinks it true that sanitation is still and always will be the foundation of sound public health. Before the days of Pasteur, an unclean environment was thought to be a direct cause of disease; it is now known that the effect is not direct, but indirect. Living organisms, not the dirt, are the active agents. Yet, the influence of dirt itself is there and must not be neglected. To give a single example, the incidence of tuberculosis among stone-cutters, has been found to be directly proportional to the proportion of silica in the stone-dust. American health workers in France have stated that it is almost impossible to instill into the minds of the people the import-

ance of living hygienic lives, if in the cities in which they live no adequate attention is paid to the disposal of fecal matter, sink wastes, animal manure, and household refuse. The speaker's students have written about their work and from everywhere the same story is told—sanitation is the foundation of health reform. We cannot have a sound mind in a sound body unless we have clean bodies in a clean world.

Administration of Sanitation

It is important not to confuse the legal status of sanitation with its governmental administration. Granted that sanitation affects health and that even minor matters of sanitation may be justified as health measures, it does not necessarily follow that all sanitary activities should be carried out by the boards of health. Prior to the days of applied bacteriology, the control of sanitation was the traditional and almost the sole function of the board of health. In recent years, the emphasis has changed from the environment to the person, from sanitation to preventive medicine and hygiene. Furthermore, government itself is changing. Force is being supplemented by service. The functions of local health boards and State health departments are accordingly being altered. To-day, the compulsory features of public health administration are being overshadowed by the service features. Laboratory diagnostications, the preparation and distribution of biological products, child welfare, infant welfare, maternal welfare, and the like are in full swing and are receiving increasing appropriations. It should be remembered that these matters are supported under the taxing power and that appropriations are justified only in so far as they are for the public benefit and as far as the results warrant the cost. Public economy, not altruistic idealism, should control governmental service. Altruism blossoms as a flower in individual lives, but dispersed by government, its growth is rank and demoralizing. Governmental attempts to benefit individual health and welfare are apt to be inordinately expensive.

At present, some of the ablest health officers are seeking to shift the burden of minor matters of sanitation, often spoken of as "nuisances", to the police departments, to turn the collection and disposal of refuse over to the department of public works, to allow the building department to administer the plumbing laws, to place smoke control under some other agency. In short, we are facing a possible administrative cleavage between the "new public health", as it is sometimes called, and sanitation.

Administrative Relationship

In health departments, as in other branches of government, there is a tendency toward the single executive type of organization. This may be merely a natural reaction from too much democracy, but it is also an indication that the public service activities in the interest of health are overshadowing the quasi-legislative and quasi-judicial functions of the older board of health. It is natural and proper that the executive of a single-headed health department should be a doctor of medicine; it is also natural that a doctor of medicine should be more interested in and more competent to handle matters of health on their personal side than to deal with matters of sanitary engineering. If the

single executive type of organization is to remain, it may be wisest to separate sanitation from the health department and, perhaps, couple it with safety, uniting the various environmental factors into a new department of government in charge of a sanitary engineer.

A serious objection to this separation would be that the people, their political representatives, and even the Courts, would gradually come to hold the idea that sanitation is distinct from health; and this would lead to a loose application of the police power in matters of sanitation and to a possible lowering of the sanitary morale of the community. On the other hand, it is not right to use the word, health, as a club to extort appropriations and secure needlessly harsh regulations, or to bolster up systems of license for plumbers or engineers.

The speaker does not favor this administrative separation, at least as far as State departments are concerned. He cannot forget the phenomenal success of the Massachusetts State Board of Health in its early days, when for a quarter of a century two men dominated its activities—one a physician, the other an engineer, Dr. Henry P. Walcott and the late Hiram F. Mills, Hon. M. Am. Soc. C. E. It seems that the proper balance between the personal and environmental factors relating to public health can best be secured if the State organization is that of a board of health, the membership of which includes not only physicians, but an engineer, a lawyer, a business man, a manufacturer, and a farmer. At present, the service activities appear to be paramount in interest and receive the largest appropriations, but any one who has served in a State department of health knows that the quasi-legislative and quasi-judicial functions are exceedingly important. In the speaker's opinion, a board composed chiefly of laymen can legislate better than an individual or a group of specialists. Perhaps, a satisfactory arrangement would be to have a State board of health with two principal departments, one of epidemiology and hygiene, and one of sanitation; the first headed by a physician trained in public health, the other headed by a sanitary engineer also trained in public health; the first taking care of health on its personal side, the other dealing with environmental problems in the interest of health. A subdivision of a health department which makes sanitary engineering merely one of half a dozen divisions unduly minimizes the importance of the environment as a health factor, and leads to an illogical budget and an inferior engineering personnel.

In cities, the sanitary activities, if removed from the local board of health, may have to be scattered somewhat in the interest of economical operation and public convenience, yet as far as possible they should be kept together. Cities differ in size and a distribution of functions appropriate to a city of 50 000 people might be quite inappropriate to a city of 500 000. There might be economy, for example, in placing refuse collection, street cleaning, paving, etc., in a street department or a department of public works. It would be a convenience to prospective builders if all building permits, covering construction matters, plumbing, etc., could be concentrated under one head.

The main thing to keep in mind is the principle that health is dependent on two factors: Man and his environment. The pendulum must not be allowed

to swing too far in either direction. The speaker is certain that an engineer is well within his prerogative when he counsels equilibrium in the various activities which have to do with health and life.

Present Status of Sanitary Engineering

The present status of sanitary engineering is in some respects not what it should be. In the Army during the World War, matters of sanitary engineering were divided between three departments: The Engineer, the Quartermaster, and the Medical. In the Medical Department, the Sanitary Corps was a catch-all and not a well-organized branch of the service. Its officers did not have rank and grade commensurate with their service or an equality with the medical officers. In the U. S. Public Health Service, the engineers do not now have the same status as the medical men. It is time that leaders in education, in public health, and in municipal government gave serious consideration to the re-organization of the Sanitary Service. It is time that the Federal Government took steps to re-organize the sanitary work of the U. S. Public Health Service and the Army and the Navy. This can be done better now, in times of peace, than at some future time when an emergency arises.

A personnel bill to promote the efficiency of the Public Health Service, which among other things authorizes the commissioning of sanitary engineers and gives them rank, grade, and pay equivalent to medical officers, has been prepared by the Treasury Department and presented to Congress for consideration. Engineers should stand solidly behind this measure as a first step in promoting the status of sanitary engineering in the U. S. Public Health Service. With engineers regularly commissioned, it will be easier to organize them into a separate section or division of the Service, a second step which should soon follow the first. At present, there are only about twenty-five sanitary engineers in this Service; there is no sanitary engineering branch of the Hygienic Laboratory; no engineer on its Advisory Board. Yet, there are far-reaching sanitary problems which should be receiving attention at Washington, problems of the pollution of interstate streams and coastal waters, problems of water purification and the protection of ground-water supplies, problems of refuse disposal, air pollution, and the like. The speaker does not mean an extension of Federal control over these matters—believing that State administration, State co-operation, and organization of river districts offer better solutions—but scientific studies carried on in a way that can be done only by the Federal Government, with ample means and an able personnel. The U. S. Public Health Service is now not well balanced as between medical officers and engineers.

Another example of unbalance may be seen in the American Public Health Association, the Association which, par excellence, represents public health in this country. Although it has an active Section of Sanitary Engineering, there is now only one practicing sanitary engineer among its forty-five elected Counsellors; in 1923, there was none. In the programs of the general sessions of the annual meetings, there have been practically no papers or addresses dealing with problems of environment. The blame for this situation lies with the

engineers as well as with the medical officers. Sanitary engineers fail to take an interest in the personal side of health protection, and medical health officers are too much engrossed in that side to give deserved attention to the environmental side.

The International Health Board of the Rockefeller Foundation has no engineer member. The Health Committee and the Health Section of the Secretariat of the League of Nations have no engineer members. Sanitary engineering in the State departments has been better recognized, and the good results are evident. In cities, there is much to be desired, but before local administrative plans are determined, the main question should be given thorough discussion: Shall public sanitation be separated administratively from public health, or shall there be co-ordination on an equal footing?

The number of students electing sanitary engineering in engineering schools is at low ebb. It is difficult to obtain enough well educated young engineers. Unless more students enter this field, there will soon be a serious shortage. The speaker will not attempt to explain why students are looking toward business and industry rather than public service, or why they are seeking positions where their work is paid for by fees rather than by salary. These are general tendencies not confined to sanitary engineering. It is probable that, in the past, sanitary engineering programs were poorly constructed, that the attempt to educate sanitary engineers by combining engineering, chemistry, and biology in substantially equal parts was a mistake. A sounder method is to develop the civil engineer first and then lead him into the special field of sanitary engineering.

The status of sanitary engineering has been referred to, not because sanitary engineers are dissatisfied with their lot—few of them are—but because the public health movement of to-day needs the steadying influence of the engineer, needs his critical, logical, calculative mind, his powers of construction and organization, and his ability to determine costs and relative values.

Life

Health is not life, but a state, quality, or condition of life. Sanitation, hygiene, and all other health agencies are but means to an end. Their chief purpose should be to develop richer and fuller individual lives and a higher civilization. To save life is not enough. What is the gain in saving the lives of children in the slums, if they whose lives are saved must always live in slums? Human happiness and usefulness, not mere longevity, should be the goal. It may be argued with much force that the health promotive agencies which act positively and contribute to fullness of life are really more important than those protective agencies which act negatively and merely prolong life without advancing it to a higher plane. Let it be remembered that throughout a man's life until old age sets in the chance of living is greater than that of dying. The quality of life is of more account than the event of death.

This is said to be a material and pleasure-seeking age. At the same time, it is called the age of the engineer, the age of power. If these statements are

true, a great responsibility rests on the engineer. There is a strong tendency for engineers to lose sight of the great elements of life. The time-honored professions of theology, law, and medicine are fundamentally personal in the service they render; the Engineering Profession is not only newer than the others, but deals chiefly with materials and natural forces, and its benefits are collective rather than individual. Engineering will be recognized as a profession only as engineers give personal disinterested service and as they make people realize the close relation of their work to life, liberty, and the pursuit of happiness.

Looking back a century or more, the beginnings of industrialism, the rise of the factory, and the growth of cities can be seen as a result of science and engineering; one can see the early evils of industrialism leading to a humanitarian movement, the great sanitary awakening at the beginning of the Victorian era. Then came Pasteur, the science of bacteriology, and the new public health. Now, with advanced ideas of industrial humanics, housing, and city planning, we are entering on a new era of sanitation, and once more the engineer must lead. Industrial revolution and a rude economic awakening are already upon us. The age of power is becoming the age of super-power; city planning has already become regional planning; decentralization of population is coming. The new problem is not so much how to educate a few sanitary engineers, but how to educate all engineers to work in the interest of health and life.

The sanitary engineer especially comes close to life. A knowledge of biology is fundamental to his success. His work is a contribution to the great elements of life. He has opportunities to protect and promote health and comfort. He has opportunities to protect the beauty of trees, streams, lakes, and ocean shores, to build beautiful structures, to utilize the latent beauty of falling water. Although burying much of his work under ground, he makes possible the development of noble city plans. Through organized cleanliness, he makes it possible for the beauties of architecture to be revealed. The new architectural roof motive, already evident in New York, resulting from the restriction of building heights is an outgrowth of a sanitary demand for adequate light and air. Through insistence on individual responsibilities in public sanitation there is being developed a moral sense of duty and a spirit of co-operation, which is the very soul of civilization.

It is well, at times, for sanitary engineers to stop and look at these things, and even to indulge in a little self-conscious professional pride, in order to encourage young men to enter this field of work. Sanitary engineering is a worthy, beneficent calling; it offers a sufficient living, opportunities for scientific achievement in varied fields, and many pleasures. Those who began work at the time the spirit of Pasteur was stimulating all public health efforts, have witnessed striking changes in living conditions, due to improved sanitation of air, soil, water, food, streets, houses, factories, and public buildings. They have seen pestilences yield to science; the art of water purification accelerate until a grossly polluted public water supply is now almost a thing of the past; and the art of sewage treatment push toward the goal, but with many yards yet to go. They have seen city and regional planning develop as the art

which is to save cities from their own colossal folly. They have seen chemistry, biology, and engineering unite in solving sanitary problems. They are now seeing young men from all parts of the world coming to America to learn the arts of sanitation and health promotion and returning to their several countries to practice them and teach them in their own universities. World-wide sanitation is looming ahead as a Twentieth Century possibility, for an age is fast approaching in which the unit of environment is the world.

DISCUSSION

RUSSELL SUTER,* M. AM. SOC. C. E. (by letter).—Life is usually described as a struggle against environment. This description places undue emphasis on the adverse effects of environment, for, although life is terminated by the adverse, it is maintained by utilizing the favorable elements.

Health is the condition of being attuned to the environment. To maintain this condition, man particularly utilizes the services of two groups of specialists: The physician, whose duty it is to repair the damage caused by unfavorable environmental elements and to fit man to his environment; and the sanitary engineer, whose province is to change the environment to fit the man.

Public health-regulative bodies, the duty of which is to promote health and increase the span of life, must utilize the services of both these groups. Possibly the functions of the sanitary engineer in public health are, from their nature, of greater magnitude and scope than those of the physician. The latter primarily deals with individuals, or private matters, whereas the former handles matters of more general public import.

It is perhaps unfortunate that in the public mind anything relating to health has become closely connected with the physician, whose principal function is to restore health rather than to preserve it. Nevertheless, it seems to the writer that the term, "health", as applied to public departments, is a proper one. It must be recognized as fundamental that the activities of the sanitary engineer are at least as necessary to the administration of public health as those of the physician.

Unfortunately, the training, mental habits, and practices of the physician and the engineer are so at variance that they do not work well together and either is prone to assume an attitude of contempt toward the other. This condition shows itself in the discussion as to whether the head of a public health department should or should not be a physician. The writer does not consider that the requirement that a public health officer should be a doctor of medicine is wise, but he would not go so far as to state that the use of a physician in this position should be forbidden. It is an administrative office, the requirements for which should be administrative ability. Neither a physician nor a sanitary engineer can be a successful health administrator without proper preliminary training in that work. If he has suitable administrative capacity, he will know enough to assign technical questions to his technical advisers for settlement and to be guided by their opinions.

The writer would consider that administrative cleavage between public health and public sanitation would be most unfortunate, although in making that statement he appreciates that present conditions are far from ideal.

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Paper No. 1553

ANALYTICAL SOLUTION OF MASONRY DOMES*

By David C. Coyle,† M. Am. Soc. C. E.

With Discussion by Messrs. Charles S. Whitney, William Cain, and David C. Coyle.

Synopsis

The object of this investigation was to develop analytical formulas for pyramoidal and conoidal masonry domes, analogous to the equations which have been established for spherical and conical shapes. The usual method of considering the forces on an elementary voussoir was adopted; that is, all the factors which may vary independently were kept separate, and appear in the final equations. These equations give the meridional and ring stresses at any point in any dome the plan of which is radially symmetrical. There is also an expression for the strength required in the tension ring at the top of a cylindrical support for a dome. The formulas have this advantage, that the designer, once having assented to them, can comfortably forget how they were derived.

Having developed these general equations, it was necessary to justify them by showing that the established formulas for special cases can be derived from them. This proves to be true in every instance.

Finally, the general equations give occasion to discuss certain facts which have been overlooked or confused in some of the previous writings on the subject. The paper aims to show that the conclusions derived from the formulas are those which might be expected and involve no *reductio ad absurdum.*

The following discussion deals with the stresses in a dome of any profile that is symmetrical about the vertical axis. The formulas submitted are

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simpler than any which the writer has seen, and it will be shown that they are consistent with the established equations for spherical and conical domes.

In all cases, the line of stress is assumed to lie in the middle of the section as in most modern treatments of the subject. As domes are usually monumental, the increased factor of safety is no detriment, whereas there is the added advantage of analytical formulas which are simple enough to be of practical use. The formulas apply approximately to any line of stress whether in the center or not.

Notation

In the following notation (Fig. 1), units are given in pounds and feet:

A = meridional stress, in pounds per foot of horizontal section.
B = ring stress, in pounds per foot of profile.
C = tension in the top ring of a vertical support, or compression in the bottom ring of a vertical lantern, in pounds.
W = sum of all loads above a given level, x.
p = average weight of shell per square foot at level, x.
R = radius of curvature of profile at x.
r = horizontal distance from x to axis of dome.
θ = angle between radius, R, and the horizontal.
H = radial bursting force, in pounds per square foot, on a circle of radius, r, for an area of 1 ft. of circumference $\times$ 1 ft. of profile.

The following applies to pyramoidal domes:

R = radius of curvature of the profile on a diagonal, at level of x.
D = semi-diagonal of horizontal section at x.
d = semi-diameter of horizontal section at x.
n = number of sides of polygonal horizontal section.
L = side of polygon, horizontal section at x.
ϕ = angle between diagonal and side of polygon.
M = bending in a horizontal element at level, x, in foot-pounds per foot of profile.

The following notation applies to spherical domes:

S_0 = height from top of masonry to point, x.
p = uniform weight per square foot of shell.
h = height from center of sphere to point, x.
V = weight of lantern, in pounds.
w = weight per cubic foot of shell.

t = thickness of shell $= \frac{p}{w}$

The following applies to conical domes:

a = vertical height of apex of cone above x.
h = vertical height of top of masonry above x.

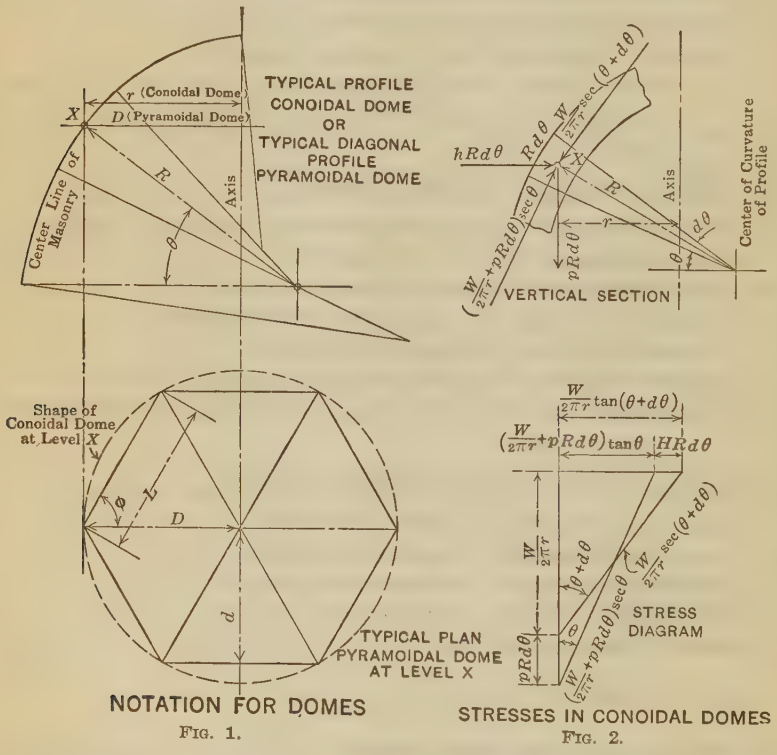
s = slant height of top of masonry above x.
r_0 = horizontal radius at top of masonry.
p = uniform weight of shell per square foot.

Conoidal Domes

Fig. 2 shows a vertical section of part of a conoidal dome, that is, a dome generated by revolving any profile about a vertical axis. The forces used are those acting on a voussoir, 1 ft. long and $R\,d\,\theta$ high, at any point, x.

The following forces act on the voussoir: The meridional thrust from above $= \frac{W}{2\,\pi\,r} \sec(\theta + d\,\theta)$; the horizontal component $= \frac{W}{2\,\pi\,r} \tan(\theta + d\,\theta)$; and the vertical component $= \frac{W}{2\,\pi\,r}$. The weight of the voussoir $= p\,R\,d\,\theta$.

The meridional thrust from below (assumed to be tangent to the profile) $= \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right) \sec\theta$; the horizontal component $= \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right) \tan\theta$; and the vertical component $= \frac{W}{2\,\pi\,r} + p\,R\,d\,\theta$.



The radial force necessary to keep the thrust in the center of the shell, as assumed (sum of all the other horizontal components) $= H\,R\,d\,\theta$. This force represents the tendency of the dome to burst.

Then:

$$H\,R\,d\,\theta = \frac{W}{2\,\pi\,r}\tan\,(\theta + d\,\theta) - \left(\frac{W}{2\,\pi\,r} + p\,R\,d\,\theta\right)\tan\,\theta \ldots\ldots (1)$$

Now,

$$\tan\,(\theta + d\,\theta) = \frac{\tan\,\theta + \tan\,d\,\theta}{1 - \tan\,\theta\,\tan\,d\,\theta}$$

and,

$$H\,R\,d\,\theta = \frac{W}{2\,\pi\,r}\,\frac{\tan\,d\,\theta + \tan^2\theta\,\tan\,d\,\theta}{1 - \tan\,\theta\,\tan\,d\,\theta} - p\,R\,d\,\theta\,\tan\,\theta \ldots\ldots (2)$$

Now, let $d\,\theta$ approach 0; then,

$$\tan\,d\,\theta = \frac{R\,d\,\theta}{R} = d\,\theta$$

and,

$$H\,R = \frac{W}{2\,\pi\,r}\,\frac{1 + \tan^2\theta}{1 - 0} - p\,R\,\tan\,\theta$$

whence,

$$H = \frac{W}{2\,\pi\,r\,R}\,(1 + \tan^2\theta) - p\,\tan\,\theta \ldots\ldots (3)$$

in pounds per square foot.

Being a radial pressure per foot of circumference, H is analogous to the pressure in a pipe, and the horizontal tension, B, required to resist the pressure, H, is $H\,r$ lb. per ft. of profile.

$$B = \frac{W}{2\,\pi\,R}\,(1 + \tan^2\theta) - p\,r\,\tan\,\theta = \frac{W}{2\,\pi\,R}\,\sec^2\theta - p\,r\,\tan\,\theta \ldots . (4)$$

in pounds per foot of profile.

Also, $d\,\theta$ being 0, the meridional thrust is:

$$A = \frac{-\,W}{2\,\pi\,r}\,\sec\,\theta \ldots\ldots (5)$$

in pounds per horizontal foot.

Consider the lower part of the dome to be removed, including the voussoir ring, $R\,d\,\theta$, and replaced by a cylindrical vertical support. The dome will exert on this support a sloping force, per foot of circumference, of $\frac{W}{2\,\pi\,r}$ sec $(\theta + d\,\theta)$; or, if $d\,\theta$ is 0, this force will be $\frac{W}{2\,\pi\,r}$ sec θ, the vertical component of which is the weight of the dome, $\frac{W}{2\,\pi\,r}$, in pounds per foot of circumference, and the horizontal component of which is $\frac{W}{2\,\pi\,r}\tan\,\theta$, in pounds per foot of circumference.

The corresponding ring tension in the top of the cylinder is this horizontal thrust $\times\,r$, or,

$$C = \frac{W}{2\,\pi}\,\tan\,\theta \ldots\ldots (6)$$

in pounds.

At the top of an open dome, a cylindrical load, such as a lantern, may be placed. This load requires a compression ring in the bottom, analogous to the tension ring which has just been considered.

The load being W, it is upheld by a series of sloping forces the value of which is $\frac{W \sec \theta}{2 \pi r}$ per foot, and the vertical component of which is the load, W, or, $\frac{W}{2 \pi r}$ per foot, and the horizontal component of which is $\frac{W}{2 \pi r} \tan \theta$.

The corresponding ring compression is, therefore,

$$C = \frac{W}{2 \pi} \tan \theta$$

Equations (4), (5), and (6) are general for all conoidal domes. The corresponding expressions for pyramoidal domes, of which the conoid is a special case, will now be derived.

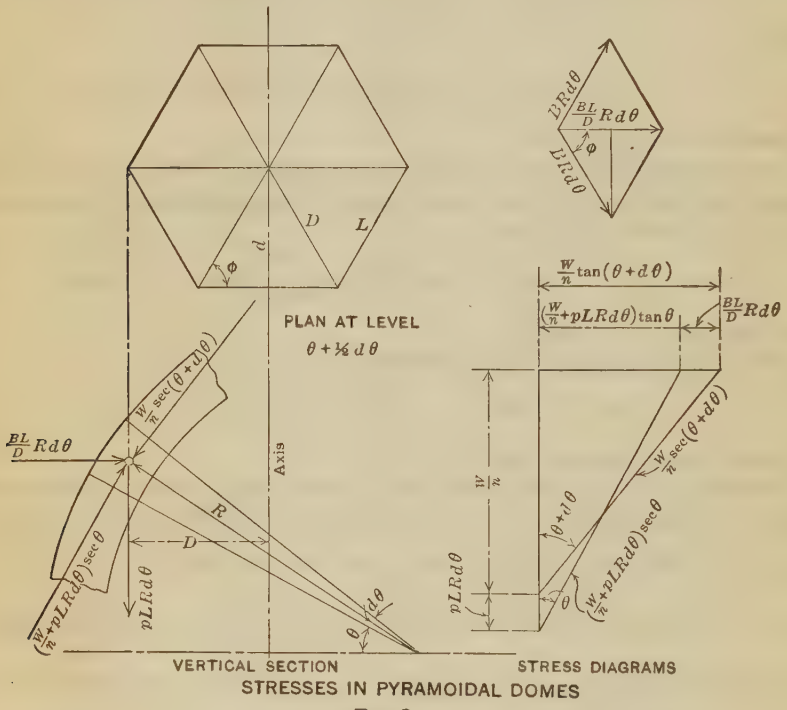


Fig. 3.

Pyramoidal Domes

The pyramoidal domes, to which these formulas apply, are characterized by a curved profile and horizontal sections which are regular similar polygons.

In Fig. 3 is shown a plan of a pyramoid, and a vertical section through the intersection of two adjacent faces. The loads are assumed as acting along the corner ribs, whether the filling between is of glass or masonry, and are shown for one rib only.

If $B\,R\,d\,\theta$ is the ring tension, the radial resultant is $2\,B\,R\,d\,\theta\,\cos\,\phi$; $\cos\,\phi = \frac{L}{2\,D}$. The radial bursting force then $= \frac{B\,R\,L}{D}\,d\,\theta$ = the difference between the horizontal components of the meridional thrusts above and below the voussoir.

$$\frac{B\,R\,L}{D}\,d\,\theta = \frac{W}{n}\tan\,(\theta + d\,\theta) - \left(\frac{W}{n} + p\,L\,R\,d\,\theta\right)\tan\,\theta$$

$$= \frac{W}{n}\left[\frac{\tan\,\theta + \tan\,d\,\theta}{1 - \tan\,\theta\,\tan\,d\,\theta} - \tan\,\theta\right] - p\,L\,R\,d\,\theta\,\tan\,\theta \ldots\ldots\ldots (7)$$

As before, with $d\,\theta = \tan\,d\,\theta$ approaching 0,

$$\frac{B\,R\,L}{D} = \frac{W}{n}\,\frac{1 + \tan^2\,\theta}{1 - 0} - p\,L\,R\,\tan\,\theta$$

$$B = \frac{W\,D}{n\,L\,R}\,(1 + \tan^2\,\theta) - p\,D\,\tan\,\theta \ldots\ldots\ldots\ldots (8)$$

in pounds per foot of rib profile.

Similarly, as $d\,\theta$ becomes 0, the meridional stress, A, equals

$$\frac{-\,W}{n}\,\sec\,\theta \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in pounds per rib.

The horizontal component of the meridional thrust, $A = \frac{W}{n}\tan\,\theta$, and the corresponding ring tension, C, in a prism support $= \frac{1}{2}\,\frac{W}{n}\tan\,\theta\,\sec\,\phi$.

$$C = \frac{W\,D}{n\,L}\tan\,\theta \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

in pounds. Equation (10) applies equally to a lantern, giving the ring compression in its lowest course.

Where the faces between the ribs are filled with solid masonry, the meridional stress along the center line of the face produces a bursting stress which is not directly resisted by the ring stress, and, therefore, must be carried in bending to the ribs. The total horizontal bursting force on any sector is $\frac{B\,L}{D}$ per foot of profile. If the masonry is continuous around the whole dome, the horizontal moment at the center of each sector and at each rib will be about, $\pm\,M = \frac{B\,L^2}{12\,D}$.

Referring to Equation (8):

$$\pm\,M = \frac{W\,L}{12\,n\,R}\,(1 + \tan^2\,\theta) - p\,\frac{L^2}{12}\tan\,\theta \ldots\ldots\ldots\ldots (11)$$

in foot-pounds per foot of profile. The fiber stresses due to this bending must be added to that due to B, which is a direct stress in the same direction.

In the case of the conoid, $D = d = r$; $n\ L = 2\ \pi\ D$; $L = 0$; $\frac{1}{n} = 0$. Equations (8), (9), and (10) reduce to Equations (4), (5), and (6), respectively, while Equation (11) becomes $M = 0$.

Stresses at Crown.—At a point near the top of any closed dome, the following conditions prevail: Area of surface of cone of base radius, r, = perimeter of base $\times \frac{1}{2}$ slant height $= 2\ \pi\ r \left(\frac{r}{2 \sin \theta}\right) = \frac{\pi r^2}{\sin \theta}$; and $W = \frac{p \pi r^2}{\sin \theta}$.

$$A = \frac{-W}{2 \pi r} \sec \theta = \frac{-p r}{2 \sin \theta} \sec \theta \quad \ldots\ldots(12)$$

$$B = \frac{W}{2 \pi R} \sec^2 \theta - p\ r \tan \theta = \frac{p\ r^2}{2\ R \sin \theta} \sec^2 \theta - p\ r \tan \theta \quad \ldots\ldots(13)$$

In the general case, the center of curvature is not on the axis, the dome is pointed at the crown, R is not vertical, and none of the functions of θ approaches 0 toward the top. A and B, being direct functions of r, = 0 at the top where $r = 0$.

Where the dome is rounded at the crown, the center of curvature is on the axis, and θ is 90° at the top. Then, $\sec \theta = \frac{R}{r}$, $\tan \theta = \frac{R}{r}$, $\sin \theta = 1$.

$$A = \frac{-p\ r}{2} \frac{R}{r} = -p \frac{R}{2} \quad \ldots\ldots(14)$$

$$B = \frac{p\ r^2}{2\ R} \frac{R^2}{r^2} - p\ r \frac{R}{r} = -p \frac{R}{2} \quad \ldots\ldots(15)$$

In no case is there any infinite stress at the top, as some writers have surmised.

Simpler Types

The general conoidal formulas will now be applied to the simpler types of dome. Fig. 4 shows one with a spherical surface.

Spherical Dome.—The area of the surface the depth of which is $S_0 = 2\ \pi\ R\ S_0$; $W = 2\ \pi\ R\ S_0\ p$; $\tan \theta = \frac{h}{r}$; and $\sec \theta = \frac{R}{r}$.

From Equation (4):

$$B = \frac{W}{2\ \pi\ R} \sec^2 \theta - p\ r \tan \theta = \frac{R^2}{r^2} S_0\ p - p\ h \quad \ldots\ldots(16)$$

From Equation (5):

$$A = -\frac{W}{2\ \pi\ r} \sec \theta = -\frac{R^2}{r^2} S_0\ p \quad \ldots\ldots(17)$$

From Equation (6):

$$C = \frac{W}{2\ \pi} \tan \theta = \frac{R}{r} S_0\ p\ h \quad \ldots\ldots(18)$$

Equations (16) and (17) have been published previously,* but Equation (18) is, the writer believes, original.

* *Transactions*, Am. Soc. C. E., Vol. LII (1904), pp. 269 and 312.

If a lantern weighs V lb., and the surface of the dome is t ft. thick and weighs w lb. per cu. ft., from Equation (4):

$$B = \frac{2\pi R w t S_0}{2\pi R}\frac{R^2}{r^2} + \frac{V}{2\pi R}\frac{R^2}{r^2} - w t r \frac{h}{r}$$

$$= w t S_0 \frac{R^2}{r^2} + \frac{V R}{2\pi r^2} - w t h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(19)$$

Equation (19) was derived and used by William Cain, M. Am. Soc. C. E.*

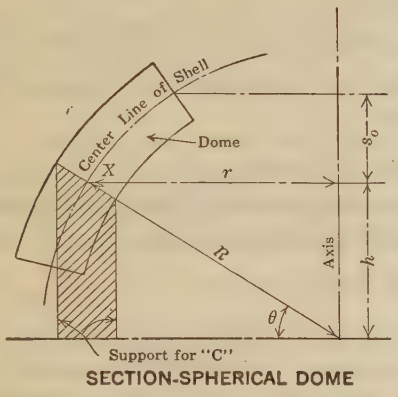


FIG. 4.

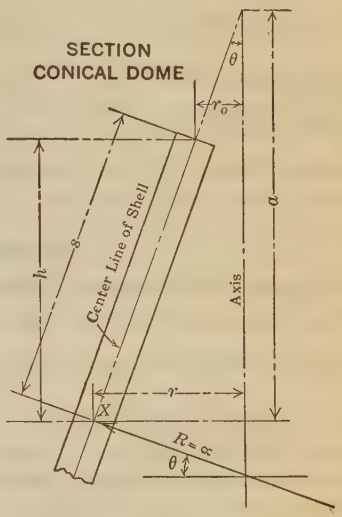


FIG. 5.

Conical Dome.—Fig. 5 shows a conical dome, for which the surface $= s\pi(r + r_0)$; $W = s\pi p(r + r_0)$; $R =$ infinity; $\frac{1}{R} = 0$; $\tan\theta = \frac{r}{a}$; and $\sec\theta = \frac{s}{h}$.

From Equation (4):

$$B = \frac{W}{2\pi R}\sec^2\theta - p r \tan\theta = -\frac{p r^2}{a} \ldots\ldots\ldots\ldots\ldots\ldots(20)\dagger$$

compression, in pounds per foot of profile.

From Equation (5):

$$A = -\frac{W}{2\pi r}\sec\theta = -\frac{p s^2}{2h}\left(\frac{r + r_0}{r}\right) \ldots\ldots\ldots\ldots\ldots\ldots(21)\ddagger$$

From Equation (6):

$$C = \frac{W}{2\pi}\tan\theta = \frac{s p r}{2a}(r + r_0) \ldots\ldots\ldots\ldots\ldots\ldots(22)$$

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* *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 205, Equation (4).

† *Loc. cit.*, p. 225, Equation (9).

‡ *Loc. cit.*, Vol. LII (1904), p. 303, Equation (73).

If the cone is closed at the top, $r_0 = 0$:

$$B = -\frac{p\,r^2}{a} \quad \ldots\ldots (23)*$$

$$A = -\frac{p\,s^2}{2\,a} \quad \ldots\ldots (24)\dagger$$

$$C = \frac{s\,p\,r^2}{2\,a} \quad \ldots\ldots (25)$$

Cylindrical Wall.—If the cone becomes a cylinder: $\frac{1}{a} = 0$, $h = s$, and $r = r_0$. From Equation (20), $B = 0$, that is, there is no horizontal stress, and from Equation (21), $A = -\frac{p\,h^2}{2\,h} \times \frac{2\,r}{r} = -p\,h$, which is the vertical load per foot of circumference. From Equation (22), $C = 0$, that is, no tension ring is required under the bottom of a cylinder.

Remarks on Formulas

A number of interesting conclusions may be drawn from these equations. It is inaccurate to state that a full hemispherical dome has no tension around the bottom; Equation (4) shows that for $\tan\theta = 0$, $B = \frac{W}{2\,\pi\,R}$, which is the greatest tension anywhere in the dome. On the other hand, $C = 0$, showing that the surface on which the dome rests receives a vertical load, and that no tension ring is required under the bottom course. The reason for this sudden change is that the curvature of the profile changes at this point from $\frac{1}{R}$ to 0. It is the curvature of the profile which causes the tension—a sort of toggle action; and where this curvature changes to zero, the tension does likewise. In practice, of course, the thickness of the stone is such that the first few courses act as a conical surface, and no such sudden change occurs.

It is sometimes noted in discussions of this subject that the horizontal tension at any point is equal to the meridional stress and is of opposite sign. This is true only for a sphere, and for a point where the meridional stress and the horizontal stress act in planes at right angles to each other. For although the parallels and the meridians on a sphere always cross at right angles, if any finite element is acted on by the forces along these lines, the resultant of the two meridional thrusts will be nearly radial to the profile, and that of the two horizontal tensions will be horizontal, the third balancing force being the weight of the element. All three are in a vertical plane. The triangle of forces is a right triangle and the horizontal and radial sides never can be equal except at the equator; for as the element approaches zero, the resultant of the meridional stresses approaches a radial position, and only at the equator is this radius in the same line with the horizontal resultant of the ring stresses. Accordingly, when $\tan\theta = 0$, $\sec\theta = 1$, and $R = r$:

* *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 314, Equation (71*a*).

† *Loc. cit.*, p. 314, Equation (70*a*).

From Equation (4):

$$B = \frac{W}{2\pi r} - 0$$

From Equation (5):

$$A = -\frac{W}{2\pi r}$$

The formulas also account for the peculiar case of a dome of low rise, in which there is no plane of rupture, since all the stresses are compression; yet it requires a tension ring at the support. Equation (4) gives the ring compression in the voussoirs just above the support, and Equation (6) gives the tension in the ring at the top of the support. The sudden change from compression to tension results from the fact that the slope changes suddenly at the support from an inclined to a vertical position, leaving the horizontal component of the thrust, otherwise unbalanced, to be taken by the structure below.

Another point not always recognized in analyses of domes is that, assuming that the line of stress is to be kept in the center of the shell, so that no vertical bending exists, the stresses at any level have to be provided for at that level, and

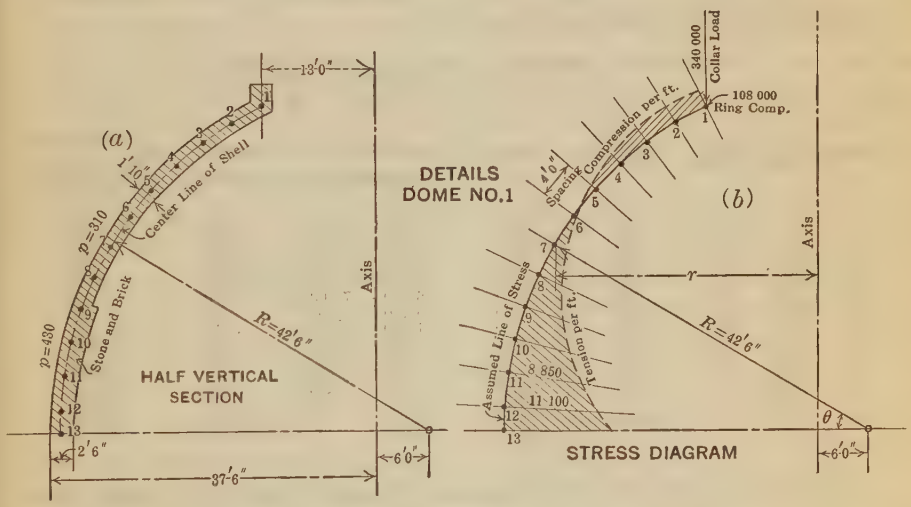


FIG. 6.

are not affected by the shape of the superstructure, nor by tension rings or abutments in the structure below. Therefore, all the zones of the dome below the plane of rupture must have their own tension rings or abutments.

This is illustrated in the case of Dome No. 1 (Fig. 6), which has tension in the lower part of the shell. The plane of rupture—an awkward name—represents the level below which the dome has ring tension. Failure will occur at any point below the plane of rupture unless there is a tension band pro-

vided at that particular point. The spacing of the tension bands, therefore, should be close enough to enable the masonry to span from one band to the next.

Where a dome is of great size and of extreme thinness the effect of unbalanced live load should be studied—a subject which will not be attempted in this paper. Under ordinary conditions, experiments seem to show that whatever the stresses may be under partial load, they do not greatly exceed at any point those due to full load. It seems reasonable to state, therefore, that for all ordinary cases the formulas herein given are sufficient for purposes of design.

Example—High Dome

To illustrate the application of the foregoing equations, two examples will be given from recent practice. Both are conoidal forms, one a high dome with a lantern, having tension in the lower part of the shell, and the other a flat dome with pure compression in the shell.

Dome No. 1 (Fig. 6 (*a*)) is a typical State Capitol dome with a heavy collar of masonry at the top, and inside this collar a lantern, separately carried, after the fashion of St. Paul's, London, England, by a conical interior dome. The stresses at various points are given in Table 1 for the exterior dome only; they were obtained from Equations (4) and (5).

TABLE 1.—Stresses in Conoidal Dome No. 1.

Point.	r, in feet.	p, in pounds.	W, in 1000 lb.	$\tan\theta$	$\sec\theta$	$\frac{W}{2\pi R}(1+\tan^2\theta)-p\,r\tan\theta = B$ in pounds per foot of profile.	$\frac{W}{2\pi r}\sec\theta = A$, in pounds per horizontal foot.
1	13.	310	340	2.0	2.23	6 400 — 8 100 = — 1 700	9 300
2	16.7	310	456	1.58	1.87	6 000 — 8 200 = — 2 200	8 200
3	19.9	310	599	1.30	1.64	6 000 — 8 000 = — 2 000	7 800
4	23.0	310	766	1.07	1.47	6 200 — 7 700 = — 1 500	7 800
5	25.8	310	956	0.90	1.34	6 400 — 7 200 = — 800	7 900
6	28.3	310	1 169	0.75	1.24	6 750 — 6 600 = + 150	8 200
7	30.6	310	1 399	0.58	1.16	7 050 — 5 500 = + 1 550	8 400
8	32.5	310	1 646	0.47	1.10	7 480 — 4 750 = + 2 730	8 900
9	34.0	430	2 006	0.36	1.06	8 450 — 3 800 = + 4 650	10 000
10	35.1	430	2 381	0.26	1.03	9 500 — 2 850 = + 6 650	11 100
11	35.8	430	2 764	0.16	1.02	10 650 — 1 800 = + 8 850	12 500
12	36.4	430	3 155	0.07	1.005	11 900 — 800 = + 11 100	13 900
13	36.5	430	3 430	0.00	1.00	12 900 — 0 = + 12 900	15 000

In Fig. 6 (*b*), the values of B shown in Table 1 have been plotted, indicating the various stresses. At the top is a ring stress due to the collar load of 340 000 lb.; $= \frac{340}{2\pi}\tan\theta = 108\,000$ lb. compression (Equation (6)).

At the springing, the ring stress, by Equation (6), $= 0$, since $\tan\theta = 0$. This means simply that this dome acts as a vertical load on a cylindrical support. Its internal stress is a different matter; at the springing, $B =$ 12 900 lb., which is clearly consistent with its values just above this point.

If the dome is cut by a vertical plane through the axis, the horizontal forces acting across the section must total zero. These forces are: The integrals of the tension and compression curve shown in Fig. 6 (*b*), about + 142

and —34, respectively; and the ring compression at the top, —108. The total is zero.

To carry these stresses, it is only necessary: (*a*) to ensure that the masonry ring at the top will carry 108 000 lb.; and (*b*) to provide tension bands capable of carrying 142 000 lb., distributed throughout the lower part of the dome in proportion to the stresses shown in Table 1. These bands must be designed for permissible stretch as well as for unit stress, and with a factor to allow for live loads, such as snow, wind, and earthquakes. These matters go beyond the scope of this paper, which is the mathematical part of the design.

It is possible to dispense with tension bands by using buttresses on the outside of the dome, or by thickening the lower part sufficiently to produce a like effect. Obviously, if the tension part of the dome was replaced with a cone tangent at Point 6, this cone would be subject only to compression. By trial, a profile could be found between this cone and the given dome, along which the ring stress would be approximately zero. This fact is of interest in monumental work in which there is an objection to the use of metal ties.

Practical Design.—The thickness of a dome of this class may be roughly fixed by several considerations. At the top, the masonry should have a minimum thickness of, say, 16 in. for stone-faced brick, 12 in. for brick, 8 in. for Gustavino, or 6 in. for reinforced concrete. The allowable stress in compression is usually assumed to be low, not more than one-half the ordinary stresses (this may require more than the thickness previously given). The size of the dome may be such as to require a greater thickness to avoid the appearance of flimsiness. When all these considerations have been met, and the thickness at the top is fixed, a somewhat greater thickness at the bottom is assumed, and the stresses are calculated; if they are less than half the usual working stresses for the material, the design may be considered to be satisfactory. For a dome of this shape, since the effect of live load is small —perhaps 20% of the total stresses—it is permissible to allow for live and unbalanced loads by keeping to a low unit stress.

Example—Low Dome

Dome No. 2 is a typical auditorium roof, with low rise and no lantern. It has no plane of rupture; the stresses are all compression. There will be no tension rings, therefore, except that at the springing.

The masonry is assumed to be 18 in. thick at the springing, varying uniformly to 6 in. at the crown. The area of the shell is about 19 500 sq. ft., and the total volume of masonry about 22 800 cu. ft. Four points will be considered, as shown on Fig. 7.

Point (*4*).—

Total weight: Masonry, 22 800 cu. ft., at 120 lb. ..... = 2 730 000 lb.
Finish and live load, 19 500 sq. ft., at 50 lb. .. = 970 000 "

Total ..... W = 3 700 000 lb.

The weight at Point (4), $p = 180 + 50 = 230$ lb. per sq. ft.; $R = 108$ ft.; $r = 72$ ft.; $\tan \theta = 0.975$; and $\sec \theta = 1.4$.

From Equation (4):

$$B = \frac{3\,700\,000}{2\,\pi\,108} \times 1.95 - 230 \times 72 \times 0.975 = -5\,500 \text{ lb. per ft. of profile.}$$

From Equation (5):

$$A = -\frac{3\,700\,000}{2\,\pi\,72} \times 1.4 = -11\,500 \text{ lb. per ft. of circumference.}$$

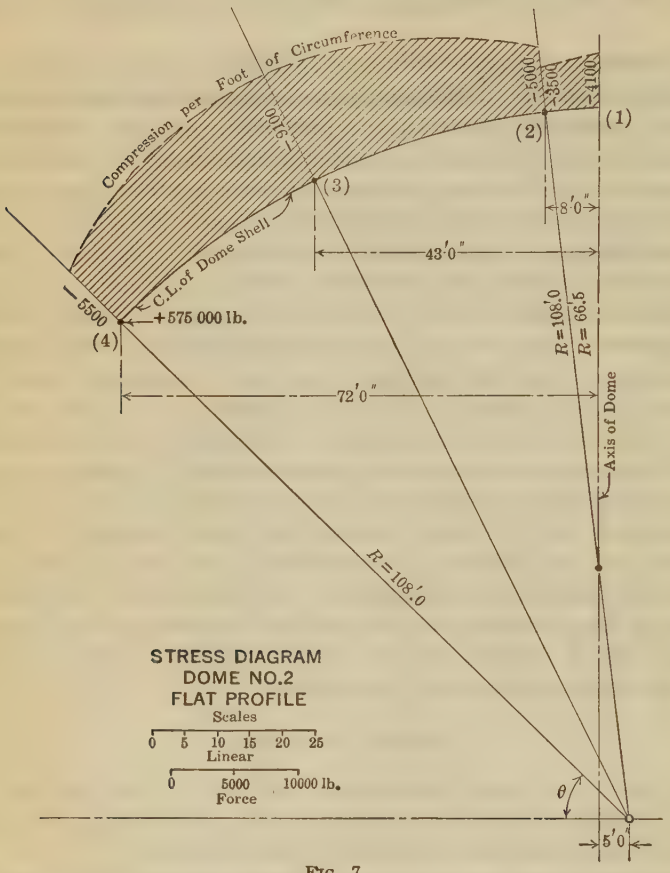


FIG. 7.

From Equation (6):

$$C = \frac{3\,700\,000}{2\,\pi} \times 0.975 = 575\,000 \text{ lb.}$$

From B and A, the masonry being 18 in. thick, the following unit stresses result:

$$\text{Ring compression} = \frac{5\,500}{12 \times 18} = 25.5 \text{ lb. per sq. in.}$$

$$\text{Meridional compression} = \frac{11\,500}{12 \times 18} = 53.5 \text{ lb. per sq. in.}$$

From C, the stress in the tension ring is 575 000 lb.

Point (3).—The volume of masonry above Level (3) is about 4 600 cu. ft., the area being about 5 900 sq. ft.

Weight: Masonry, 4 600 cu. ft. at 120 lb. . . . . = 550 000 lb.
Finish and live load, 5 900 sq. ft. at 50 lb. = 295 000 "

Total . . . . . W = 845 000 lb.

The weight at Point (3), $p = 125 + 50 = 175$ lb. per sq. ft.; $R = 108$ ft.; $r = 43$ ft.; $\tan \theta = 2.05$, and $\sec \theta = 2.25$.

$$B = \frac{845\,000}{2\,\pi\,108} \times 5.05 - 175 \times 43 \times 2.05 = -9\,100 \text{ lb. per ft. of profile.}$$

$$A = \frac{845\,000}{2\,\pi\,43} \times 2.25 = -14\,000 \text{ lb. per ft. of profile.}$$

Point (2).—At this point, the volume of masonry above is approximately, 1 170 cu. ft., the area being about 2 000 sq. ft.

Weight: Masonry, 1 170 cu. ft. at 120 lb. . . . . = 14 000 lb.
Finish and live load, 2 000 sq. ft. at 50 lb. . = 10 000 "

Total . . . . . W = 24 000 lb.

At Point (2), $R = 106$ and 66.5 ft.; $\tan \theta = 7.7$; $\sec \theta = 8.3$; $r = 8$ ft.; and $p = 73 + 50 = 123$ ft.

Substituting, as before, using both values of R:

$$B = \frac{24\,000}{2\,\pi\,106} \times 69 - 123 \times 8 \times 7.7 = -5\,000 \text{ lb. per ft. of profile.}$$

$$B = \frac{24\,000}{2\,\pi\,66.5} \times 69 - 123 \times 8 \times 7.7 = -3\,500 \text{ lb. per ft. of profile.}$$

$$A = -\frac{24\,000}{2\,\pi\,8} \times 8.3 = -3\,950 \text{ lb. per ft. of circumference.}$$

The two values of B represent the sudden change in the ring compression due to the change in radius (see Fig. 7). Point (2), of course, could not be found exactly by looking at the dome, and, similarly, no such sudden change actually occurs in the masonry. As the dome is built, the circle, of radius 108 ft., can extend some distance above Point (2) before going outside the middle third of the shell. At any rate, the formulas show that the stresses are small.

For Point (4), the stresses at the top, are given by Equations (14) and (15):

$$A = B = -p\,\frac{R}{2} = \frac{-123 \times 66.5}{2} = -4\,100 \text{ lb. per ft. of circumference}$$

or for a dome 6 in. thick, the stress is:

$$\frac{4\,100}{12 \times 6} = 57 \text{ lb. per sq. in.}$$

On Fig. 7, the values of the stresses are plotted radially to show in graphic form the variations of the ring compression. The integral of this curve is, of course, equal to the stress in the tension ring, 575 000 lb.

Theoretically, a dome of this type may have a minimum thickness of zero, except for the effect of live and unbalanced loads. Pending the development of a practical method of calculating the stresses due to such loads, it is reasonable to state that by using working stresses in compression of not more than one-half the usual amounts ample safety is secured. The usual thickness at the top varies from 3 in. for a 50-ft. dome to 6 in. for a 150-ft. dome, for tile, brick, or concrete, without distinction. At the springing, the thicknesses of some recent domes have been 3 in. and 5 in. for 24-ft. and 53-ft. concrete domes, respectively, and 7½ in. for a 98-ft. tile structure. A thickness should be assumed and the stresses calculated; if the dome is very flat, the stresses will be high, and the assumed thickness may have to be increased.

Summary

In this paper, there have been developed formulas for the ring and meridional stresses in domes, and for the stress in a cylindrical support or a cylindrical lantern, at the point of junction with the dome.

The formulas for conoidal and pyramoidal domes are here repeated in full. It is assumed that the dome will be provided with sufficient reinforcement to hold the stress line to the profile selected, which may or may not lie in the center of the masonry. The formulas apply only to symmetrical loading, and disregard elastic deformation and temperature stresses.

Formulas.—Conoids.—For conoids the following formulas hold (positive sign indicates tension):

Ring tension at level, x:

$$B = \frac{W}{2\,\pi\,R}(1 + \tan^2\theta) - p\,r\tan\theta \quad \ldots\ldots(4)$$

in pounds per foot of profile.

Meridional compression at x:

$$A = \frac{-W}{2\,\pi\,r}\sec\theta \quad \ldots\ldots(5)$$

in pounds per horizontal foot.

Tension in the top of a supporting cylinder, or compression in the bottom of a cylindrical load, applied at the point, x:

$$C = \pm\frac{W}{2\,\pi}\tan\theta \quad \ldots\ldots(6)$$

in pounds.

In a closed dome, pointed at the top like a Gothic arch, the ring stress at the crown approaches the value:

$$B = \frac{p\,r^2}{2\,R\sin\theta}(1 + \tan^2\theta) - p\,r\tan\theta \quad \ldots\ldots(13)$$

and the meridional stress approaches the value:

$$A = \frac{-p\,r}{2\sin\theta}\sec\theta \quad \ldots\ldots(12)$$

At the top of a closed dome rounded at the top:

$$A = B = -p\frac{R}{2} \quad \ldots\ldots(14) \text{ and } (15)$$

Formulas.—Pyramoids.—For pyramoids the following formulas hold:

Ring tension at level, x:

$$B = \frac{W\,D}{n\,L\,R}(1 + \tan^2\theta) - p\,D\tan\theta \quad \ldots\ldots(8)$$

in pounds per foot of rib profile.

Meridional compression at level, x:

$$A = \frac{-W}{n}\sec\theta \quad \ldots\ldots(9)$$

in pounds per rib.

Tension in the top of a supporting prism, or compression in the bottom of a prism load, applied at the point, x:

$$C = \pm\frac{W\,D}{n\,L}\tan\theta \quad \ldots\ldots(10)$$

in pounds.

Bending in a horizontal element, 1 ft. high (fiber stress due to this is to be added to B):

$$\pm M = \frac{W\,L}{12\,n\,R}(1 + \tan^2\theta) - p\,\frac{L^2}{12}\tan\theta \quad \ldots\ldots(11)$$

in foot-pounds per foot of profile.

DISCUSSION

CHARLES S. WHITNEY,* M. AM. SOC. C. E. (by letter).—The ordinary theory of domes as expressed in Mr. Coyle's paper is based on the assumption that the dome is a free body cut away from its supports, which are replaced by vertical forces and ring tension. The lantern ring, if one exists, is also considered to be free, as though both lantern and support rings bear against the dome shell without friction. Under this assumption, it is possible for a high tension or compression to exist in the ring with a low circumferential stress even of opposite sign in the adjacent shell. This, of course, is not the condition in practice, where the rings are monolithic with the dome and their deformation has an important effect on the circumferential stresses, particularly near the rings. These stresses should be considered carefully although an exact analysis of them does not seem possible.

A stretching of the supporting ring or a yielding of the abutments has the effect of increasing the circumferential tension or decreasing the circumferential compression in the shell near the supporting ring and also of increasing the circumferential compression at the top. This means that the radial component of the circumferential stress is no longer great enough to confine the meridional thrust to the center of the shell and that this thrust will rise above the center near the support and produce positive bending, which might be very dangerous in a thin shell because it has a tendency to cause bursting. The tendency is to split the lower part of the dome into radial strips which act as buttresses. Clearly these buttresses will be most effective if they are straight. It is advisable therefore to increase the thickness of the shell near the supporting ring, making it conical in shape for a short distance. The lower part of the shell should be reinforced with sufficient circumferential steel to absorb the tension induced by the ring deformation.

If the dome is exposed to the weather, temperature changes may have an important effect, as the upper part of the dome is more exposed and may pass through a much greater range of temperature than the supports. A decrease in the temperature of the portion of the dome above the supports or a shrinkage of the concrete of the shell will have the same effect as a stretching of the supporting ring. This will tend to produce compression in the supporting ring with tension in the dome shell just above.

A similar but opposite action is produced by the compression ring at the lantern unless the stress in the ring is kept as low as the circumferential stress in the adjacent shell.

In the case of the full hemispherical dome, according to the theory as stated by the author, the circumferential stress is greatest at the base while the stress in the supporting ring is zero. This of course would require that the dome be supported on a frictionless base. Actually, the circumferential deformation at the base of the shell must produce a swelling which will induce a tension in the supporting ring or compression in radial buttresses (if they are provided). The

* Cons. Engr., Milwaukee, Wis.

dome does not act as a full hemisphere but, as stated in the paper, as if the lower part were a conical surface.

For spherical domes with shells of uniform thickness Fig. 8 gives a convenient method of computing the stresses according to the ordinary theory and shows clearly the variation in their magnitude. The live load is assumed to be a full uniform load of p lb. per sq. ft. of horizontal projection.

The stresses are given by the following formulas:

Dead load stresses:

$$\text{Meridional} \ldots\ldots A_1 = A_D\, R\, g$$
$$\text{Circumferential} \ldots B_1 = B_D\, R\, g$$
$$\text{Supporting ring} \ldots C_1 = C_D\, R^2\, g\, t$$

Live load stresses:

$$\text{Meridional} \ldots\ldots A_2 = \frac{A_L\, R\, p}{t}$$
$$\text{Circumferential} \ldots B_2 = \frac{B_L\, R\, p}{t}$$
$$\text{Supporting ring} \ldots C_2 = C_L\, R^2\, p$$

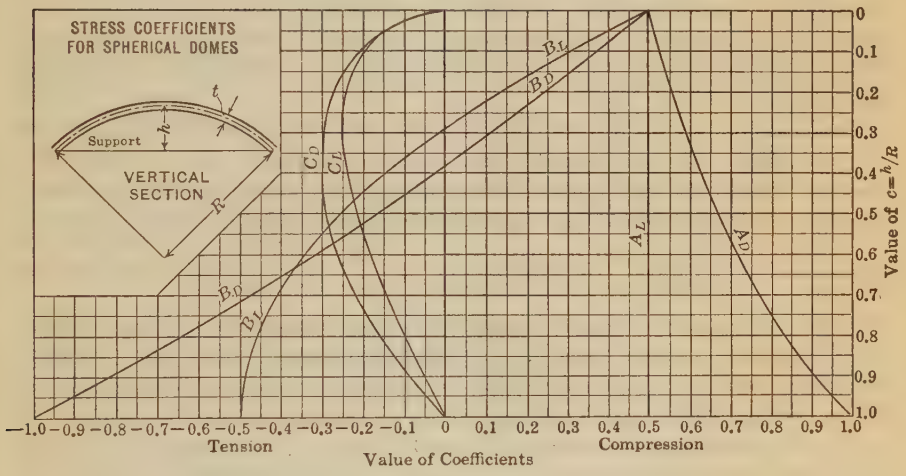


FIG. 8.

in which,

t = the thickness of the shell;

g = the heaviness of the material per unit volume; and

c = the ratio of the height of the dome to the radius of the sphere.

The values of A_D, A_L, etc., may be determined from Fig. 8.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The author is to be congratulated on his very general and simple solutions of conoidal and pyramoidal domes for the assumed line of stress. The dome is, of course, an indeterminate

* Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

structure, for which a strict solution can only be effected by consideration of the elastic yielding of its parts; but for domes of the usual thicknesses, it is thought that a practical solution can be made by assuming the line of stress to be on or near the center line of a meridian section.

The writer has already discussed rather fully such possibilities* and he agrees with the author that, using the formulas of this paper and an allowable unit stress in compression, not more than one-half that ordinarily permitted, the dome should be safe, although practical considerations usually determine the minimum thickness at the top.

To aid in comprehending the theory of the dome, a graphical treatment is a great help. The writer would call attention to his graphical method† which can be easily extended to apply to conoidal or pyramoidal domes with any assumed line of stress.

The author is somewhat ambiguous with regard to the number of forces acting on the voussoir of Fig. 2, but the four forces holding the voussoir in equilibrium are given correctly on the upper diagram. Again, with θ constant, it is seen that r varies as $d\ \theta$ approaches zero, although it is assumed to be constant in Equations (1), (2), (3), and (4). The final result is correct, but it seems best, in order to have both θ and r constant, to measure r from the center of the lower face of the voussoir; then as $d\ \theta$ diminishes indefinitely, both θ and r remain constant and X approaches indefinitely the center of the lower face.

A slightly different derivation of Equation (4), but along the lines sketched by the author, may not be amiss, as it checks his result. In Fig. 9(a), the voussoir (shown shaded) is represented as held in equilibrium by its weight, the two meridional thrusts, and the bursting pressure. In Fig. 9(b), the force diagram is shown. In this, the vertical, $O\,D = \dfrac{W}{2\,\pi\,r}$, represents the entire weight of dome, lantern, snow, etc., sustained by the upper conical face of the voussoir, and $DE = p\ R\ d\ \alpha$ represents the weight of the voussoir. The rays OA, OK, are drawn parallel to the tangents of the center line of the meridian section at the centers of the upper and lower faces of the voussoir and are limited at A and K by horizontal lines drawn through D and E. A vertical, $KI = DE$, cuts off the "bursting pressure," $AI = H\ R\ d\ \alpha$, on the horizontal, AD.

The voussoir is thus in equilibrium under the four forces, OA, AI, IK, and KO, the magnitudes and directions of which are given by the closed polygon, $O\ A\ I\ K\ O$ (Fig. 9 (b)). For equilibrium, the algebraic sum of the horizontal components of these forces must equal zero. Therefore, $AI = AD - KE$, or,

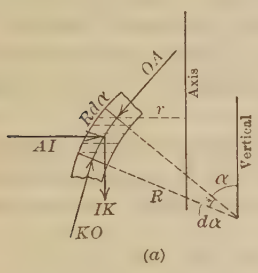
$$H\ R\ d\ \alpha = \frac{W}{2\,\pi\,r}\cot\,\alpha - \left[\frac{W}{2\,\pi\,r} + p\ R\ d\ \alpha\right]\cot\,(\alpha + d\ \alpha),$$

$$H = \frac{W}{2\,\pi\,r\,R}\,\frac{[\cot\,\alpha - \cot\,(\alpha + d\ \alpha)]}{d\ \alpha} - p\,\cot\,(\alpha + d\ \alpha).$$

* "Theory of Steel-Concrete Arches and of Vaulted Structures", pp. 156–215.

† *Transactions*, Am. Soc. C. E., Vol. LV (1905), pp. 201–227.

H, in this case, is the average bursting force per unit of area acting on the voussoir. As $d\alpha$ approaches zero indefinitely, H approaches the intensity of the bursting pressure at X, the center of the upper face of the voussoir; and, as proved in any calculus, $\frac{\cot(\alpha + d\alpha) - \cot\alpha}{d\alpha}$ approaches indefinitely $(-\operatorname{cosec}^2\alpha)$ as its limit; whence, taking limits and multiplying by r, to find the ring stress, $B = Hr$, B, the ring stress at X, $= \frac{W}{2\pi R}\operatorname{cosec}^2\alpha - pr\cot\alpha$, which is equivalent to the author's Equation (4), since $\alpha = 90° - \theta$.



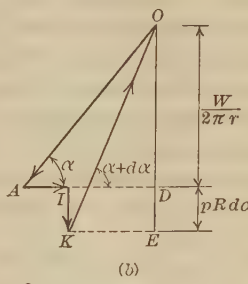


FIG. 9.

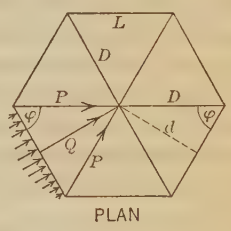


FIG. 10.

The reference to the calculus can be avoided by a little trigonometrical reduction. Thus,

$$\frac{\cot\alpha - \cot(\alpha + d\alpha)}{d\alpha} = \frac{1}{d\alpha}\left[\frac{\cos\alpha}{\sin\alpha} - \frac{\cos(\alpha + d\alpha)}{\sin(\alpha + d\alpha)}\right]$$

$$= \frac{1}{d\alpha}\,\frac{\sin[(\alpha + d\alpha) - \alpha]}{\sin\alpha\sin(\alpha + d\alpha)} = \frac{\sin d\alpha}{d\alpha}\,\frac{1}{\sin\alpha\sin(\alpha + d\alpha)}$$

the limit of which, as $d\alpha = 0$, is

$$1\,\frac{1}{\sin^2\alpha} = \operatorname{cosec}^2\alpha$$

The equivalents of the author's Equations (5) and (6) are derived directly from Fig. 9 (*b*). Thus, the meridional thrust at X is,

$$OA = A = -\frac{W}{2\pi r}\operatorname{cosec}\alpha$$

and with the interpretation given to C,

$$C = AD \times r = \frac{W}{2\pi}\cot\alpha$$

In his paper on the "Theory of the Spherical or Conical Dome of Reinforced Concrete or Metal,"* the writer used diagrams like Fig. 9 in developing formulas for meridional and crown stresses and hoop tension, corresponding in meaning to the author's A, B, and C, and he was careful to point out that the graphical method, giving average values for B (or C), would differ slightly from values found analytically, which always refer to intensity at a point. His Formulas (3), (4), and (6), for the spherical dome† agree with

* *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 201.

† *Loc. cit.*, pp. 213–217.

the author's Equations (17), (16), and (18), when the notation is made the same, a fact which the author seems to have overlooked when he states his belief that Equation (18) is original. In fact, Rankine's equivalent formula for "hoop tension" was published years ago.

In treating pyramoidal domes, Fig. 9 can again be utilized, letting $OD = \frac{W}{n}$ and $A\ I = \frac{BL}{D}\ R\ d\ \alpha$. Only a slight change in the preceding analysis is necessary to deduce the values of A, B, and C.

The author's treatment of the faces between the ribs when filled with solid masonry is open to objection. The center line of a face is not the same curve as the profile of a rib or the intersection of two faces. It is seen from Fig. 10, that $d = D \sin \phi$, so that the horizontal ordinates of the two curves at the same level are in a constant ratio. Thus, if the profile of a rib is a quadrant of a circle, the center line of a face is an ellipse.

Let Fig. 9 now refer to the face at the level of a supposed straight horizontal beam of masonry of length, L, the weight of which is $p\ L\ R\ d\ \alpha$, which replaces the weight of the voussoir, the curve of the center line of the face replacing that of the rib. Now, $OD = \frac{W}{n}$, the whole load on the face down to the level of the beam, and $DE = p\ L\ R\ d\ \alpha =$ weight of beam of height, $R\ d\ \alpha$, along the arc; then, in Fig. 9 (b), $AI = L\ H\ R\ d\ \alpha$, is the horizontal force acting at right angles to the beam at its center, as induced by the meridional thrusts. In this instance, H is the corresponding unit force, acting on a unit of length of beam and a unit of profile of the face.

From $A\ I = A\ D - K\ E$,

$$L\ H\ R\ d\ \alpha = \frac{W}{n} \cot \alpha - \left[\frac{W}{n} + p\ L\ R\ d\ \alpha\right] \cot\ (\alpha + d\ \alpha)$$

from which, proceeding as before,

$$HL = \frac{W}{n\ R} \operatorname{cosec}^2 \alpha - p\ L \cot\ \alpha$$

In this formula, HL represents the total horizontal force acting on the horizontal beam of length, L, and of one unit height along the profile of the face. The author chooses to designate this force by the awkward expression, $\frac{BL}{D}$, for which Equation (8) is identical with the writer's formula, except that the angle, α, of this formula refers to the profile of the face, whereas θ in the author's Equation (8) refers to the profile of the rib.

There are serious objections to the analysis for evaluating HL, for it tacitly assumes that the face is divided up, not by meridian planes but by vertical planes, all perpendicular to the horizontal beam, say, one unit apart. All the corresponding shells, of unit width, will have parallel profiles and parallel tangents, and their weights, for a full dome, will vary nearly linearly from a maximum at the center of the beam to zero at the ribs, and the unit horizontal forces acting on the beams will vary correspondingly according to the theory mentioned. There is now, however, shear along the divisional

planes, so that the theory does not exactly apply. The effect of this shear will be to make the unit horizontal forces acting on the beam more nearly uniform.

This is seen, also, by dividing the shell of the face by meridional planes, between which there is, presumably, no shear, as there certainly is none for a conoidal dome. If a face of a pyramoidal dome is divided by meridional planes, all passing through its vertical axis, a voussoir of the horizontal beam contained between two such planes, will be in equilibrium under its weight, the two meridional stresses, and a horizontal radial force. This last force is not perpendicular to the beam except at its center, and its component perpendicular to the beam is less at the ends than at the center. The sum of all such components acting on the beam is equal to the force, Q. The components are not uniformly distributed, but the component at the end of the beam is not zero; so that the supposition that Q varies from a maximum at the center linearly to zero at the ends will give an excess moment at the center.

Although the writer would not advise the adoption in practice of the imperfect solution just given, still it suffices to bring out the fact, not hitherto noted, that the unit forces acting on the horizontal beam are not uniform; and to give an excess moment at the center of the beam, they can be regarded as decreasing uniformly from a maximum at its center to zero at its ends.

If Q represents the sum of the horizontal forces acting on the beam corresponding to the latter hypothesis, then, if the beam is regarded as fixed at the ends, the moment at the ends is easily shown to be $-\frac{5}{4}\left(\frac{1}{12}\ QL\right)$ and, at the center, $+\frac{3}{4}\left(\frac{1}{12}\ QL\right)$. For a simple beam, supported at the ends, the moment at the center is $\frac{1}{6}\ Q\ L$. The degree of fixity at the ends must determine the choice of coefficients, which, for usual designs, may approximate $\left(\frac{1}{12}\ QL\right)$ at the center and ends, as assumed by the author.

For a pyramoidal dome of masonry, P (Fig. 10), may be regarded as the resultant of the forces, $\frac{Q}{2}$, $\frac{Q}{2}$, acting on the lengths, $\frac{L}{2}$, of the horizontal beams on adjacent sides of the perimeter.* Thus,

$$2\left(\frac{1}{2}\ Q\ \text{cosec}\ \phi\right) = P\ \therefore\ Q = P\ \sin\ \phi = P\ \frac{d}{D}$$

or, since, in the author's notation, $P = \frac{B\ L}{D}$, by the use of his Equation (8),

$$Q = \frac{d}{D}\left[\frac{W}{nR}\ \sec^2\theta - p\ L\ \tan\ \theta\right]$$

To the fiber stresses at the ends and center of the horizontal beam due to the moment of, say, $\pm\frac{1}{12}\ QL$, must be added that due to the longitudinal stress, B.

* As given by Mr. E. Schmitt in his paper entitled "Theory of the Spherical Dome with a Homogeneous Surface, and of the Framed Dome, also Notes on the Construction of Masonry and Metal Domes", *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 293.

The author has deduced the formulas for hoop tension, ring, and meridional stresses, for the conical dome from the corresponding formulas for the spherical dome, by making R equal to infinity.

All these formulas, like the corresponding ones for the conoidal and pyramoidal dome, are correct and in practical form. It is a pleasure to commend the paper as a good, brief, practical treatise on the interesting subject of the dome.

David C. Coyle,* Esq. (by letter).—Mr. Whitney calls attention to the fact that there may be bending stresses in the shell near the support and near the top, which would be dangerous for a thin shell. Where the dome has no plane of rupture, the yielding of the ring tends to straighten the profile just above it, for the parts closest to the ring will stretch the most. That is, the bending is positive, and the tendency would seem to be not to burst, as Mr. Whitney says, but to collapse downward if the profile should become so far distorted as to result in a reverse curve, concave upward. In that case the extra masonry should be on the outside rather than on the inside, near the bottom, to provide room for the line of stress to move outward without getting too near the surface.

Where the dome has tension in the lower part, the rings taking this tension will stretch about the same as the ring at the springing line, if any, and no dangerous effects on the profile are likely to occur.

At the top, the compression tends to increase the curvature of the profile; on a very flat, thin dome, it might even conceivably cause the formation of a low spot in the middle, like the end of Columbus' egg. The meridional stress would then become tension and the middle would drop out, unless held by meridional reinforcement. All these elastic effects are difficult to compute exactly, especially in masonry with an uncertain elastic coefficient, but the recognition of them is an essential part of the "judgment" that needs to be used in connection with formulas such as these, in which not all the facts are expressed.

Regarding the bending on the faces between the ribs of a pyramoidal dome, Professor Cain's criticism is well taken; the total horizontal force acting on the horizontal beam, L, is not $\frac{B\,L}{D}$, as stated in the derivation of Equation (11).

Referring to Fig. 11, the horizontal force at the rib is $\frac{B\,L}{D}$, which is composed of the reactions of the two horizontal beams, L, on each side of the rib. Each of these reactions is then:

$$\frac{1}{2}\,\frac{B\,L}{D}\,\text{cosec}\,\phi = \frac{B\,L}{2\,D}\,\frac{D}{d} = \frac{B\,L}{2\,d}$$

and the total of the forces acting on the beam, L, has $\frac{B\,L}{d}$ as its component perpendicular to the beam.

* Designing Engr., Gunvald Aus Co., New York, N. Y.

As to the distribution of this load there might be assumed a series of sectors parallel to D, which would give a load varying from zero at the center to a maximum at the ends, or a series parallel to d, giving a load varying from zero at the ends to a maximum at the center. It seems reasonable for practical purposes, then, to assume a uniform load and use the coefficient, $\frac{1}{12}$.

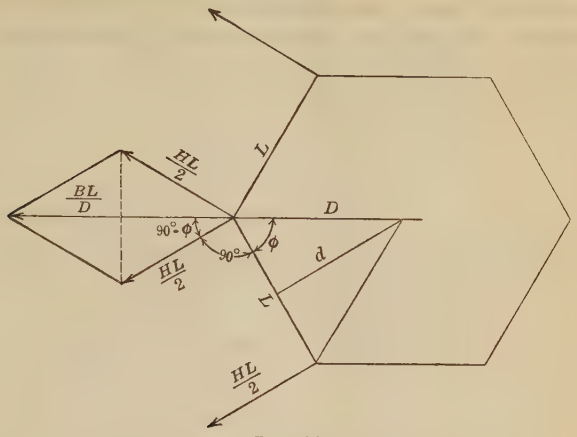


FIG. 11.

The bending would be $\frac{B\,L}{12\,d}$ instead of $\frac{B\,L}{12\,D}$, as previously given, or,

$$-M = \frac{D\,W\,L}{12\,d\,n\,R}\,(1 - \tan^2\theta) - p\,\frac{L^2\,D}{12\,d}\tan\theta$$

rather than Equation (11), as given.

Professor Cain seems to be in error in his derivation of Q, for $P = Q \sin\phi$, instead of $Q = P \sin\phi$ as he states. This error in Mr. Schmitt's paper to which he refers is the reason for the fact, pointed out in the discussion at the time, that some of Mr. Schmitt's formulas are incorrect.

The expression for the value of $H\,L$ given by Professor Cain is correct, if R is the radius of curvature of the diametral profile, and α, the angle between R and the vertical. Neither of these quantities can be correlated with the corresponding quantities for the diagonal profile, as their relation depends on the particular form of the profiles. It is possible, however, to derive a value for the ring tension, using Professor Cain's formula. Let

$$H\,L = \frac{W}{n\,S}\sec^2\beta - p\,L\tan\beta$$

in which S corresponds to R, and β to θ, in Equation (8).

Now,

$$H\,L = \frac{B\,L}{d};\; B = \frac{H\,L\,d}{L}$$

therefore, Equation (8a) will be :

$$B = \frac{W\,d}{n\,L\,S}\sec^2\beta - p\,d\tan\beta$$

This equation, although identical in general form with Equation (8), would give values of B probably slightly smaller. Equation (8a) is based on the assumption that the dome consists of a series of elements in planes parallel to d, while Equation (8) assumes elements parallel to D. The assumption of elements in radial planes would doubtless be more nearly correct, as Professor Cain remarks, but the formula is too complicated for practical use, and Equation (8), involving a longer span than the others, will probably always give the largest stresses. As low unit stresses are used in any case, the choice is mostly a matter of convenience.

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Paper No. 1554

CERTAIN EFFECTS OF BRIDGES ON TRANSIT CONDITIONS*

By John A. Miller, Jr.,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Charles Evan Fowler, M. W. Weir, Herman H. Smith, D. B. Steinman, H. M. Lewis, T. Kennard Thomson, Herbert C. Keith, Ole Singstad, and John A. Miller, Jr.

Synopsis

This paper is a study of the effect of the construction of important vehicular bridges on transit conditions in the adjacent communities. Although the primary object of building such structures is usually the accommodation of vehicular traffic, railway tracks are likely to be included, and the increased transportation facilities thus provided cause far-reaching changes in the trend of passenger traffic. In particular, the effect has been studied of the construction of the Brooklyn and Williamsburg Bridges on travel between the Boroughs of Manhattan and Brooklyn of the City of New York. As an example of the possible application of the conclusions drawn from that investigation, the situation created by the erection of the new Delaware River Bridge between Philadelphia, Pa., and Camden, N. J., has been considered. An analysis of traffic movement across the East River, New York, indicates that the existing transportation lines in New Jersey should be extended across the new bridge into Philadelphia.

Introduction

Influenced by the great increase in the use of motor vehicles during the past few years, there has been a correspondingly greater demand for vehicular

* Presented at the meeting of May 7, 1924.
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bridges. Additional facilities have been provided in many places where existing structures were inadequate, new bridges have been built where none previously existed, and there are probably more large bridges under construction or under consideration than ever before.

After an important vehicular bridge has been built across a wide river, and the communities on the two banks are thus brought more closely together, great changes may be expected in the trend of traffic. Frequently, besides the primary effect on vehicular traffic, there is also a secondary effect on local passenger transportation. In any given case, a study of the development and outcome of similar situations elsewhere, should forecast the effect that a new bridge will have on existing facilities.

An excellent example of the tremendous changes in passenger traffic which may follow the completion of an important bridge is furnished by the history of the Brooklyn Bridge. Before its construction, the only method of travel between Lower Manhattan and Brooklyn was by the East River ferries; the building of this, the first, bridge had a far-reaching effect on all local traffic, particularly that by ferry. This effect may reasonably be expected in any similar case; in fact, the construction of the Williamsburg Bridge, replacing ferry service between sections of Manhattan and Brooklyn not accommodated by the first bridge, had an almost identical effect.

However, since conditions are seldom exactly alike in any two cases, the conclusions reached from a study of East River traffic by bridge and ferry may have to be modified before estimating future traffic elsewhere.

The Brooklyn Bridge

Prior to the opening of the Brooklyn Bridge, the traffic between Lower Manhattan and Brooklyn was carried chiefly by the five lines of the Union Ferry Company. Beginning with the most northerly, these lines were, in order: (1) Catherine Ferry, from Main Street, Brooklyn, to Catherine Street, New York; (2) Fulton Ferry, from Fulton Street, Brooklyn, to Fulton Street, New York; (3) Wall Street Ferry, connecting Montague Street, Brooklyn, and Wall Street, New York; (4) South Ferry, from Atlantic Avenue, Brooklyn, to the foot of Whitehall Street, New York; and (5) Hamilton Ferry, from Hamilton Avenue, Brooklyn, to Whitehall Street. These routes are shown in Fig. 1.

Passenger Traffic.—About one-half the total passenger traffic was carried by Fulton Ferry, using four boats on a 5-min. headway. The remainder was divided about equally among the other four lines, each operating on a 10-min. headway. The ferry fare was the same on all lines—1 cent during rush hours, otherwise 2 cents. Records show that the business done by the Union Ferry Company in 1871 amounted to 41 000 000 passengers, increasing by 1883 to more than 51 000 000, as indicated in Fig. 2.

On May 24, 1883, the Brooklyn Bridge was opened, and in September of that year, the operation of the Bridge Railroad (cable) was commenced. This railroad was simply a shuttle-line across the bridge, running 2-car trains and charging a 5-cent fare. As the Bridge Railroad was less convenient than the ferries, and the fare was higher, it did not at first take away much traffic

from them. Approximately, 1 000 000 passengers were carried across the bridge in 1883, during the 3 months of operation; 8 500 000 in 1884; 17 000 000 in 1885; and 24 000 000 in 1886. At the same time, the pedestrian traffic increased to about 2 000 000 per year. The surprising feature of the situation, however, is that in spite of the fact that 26 000 000 people used the bridge in 1886, the ferry traffic remained at 50 000 000, or nearly as much as before the bridge was opened.

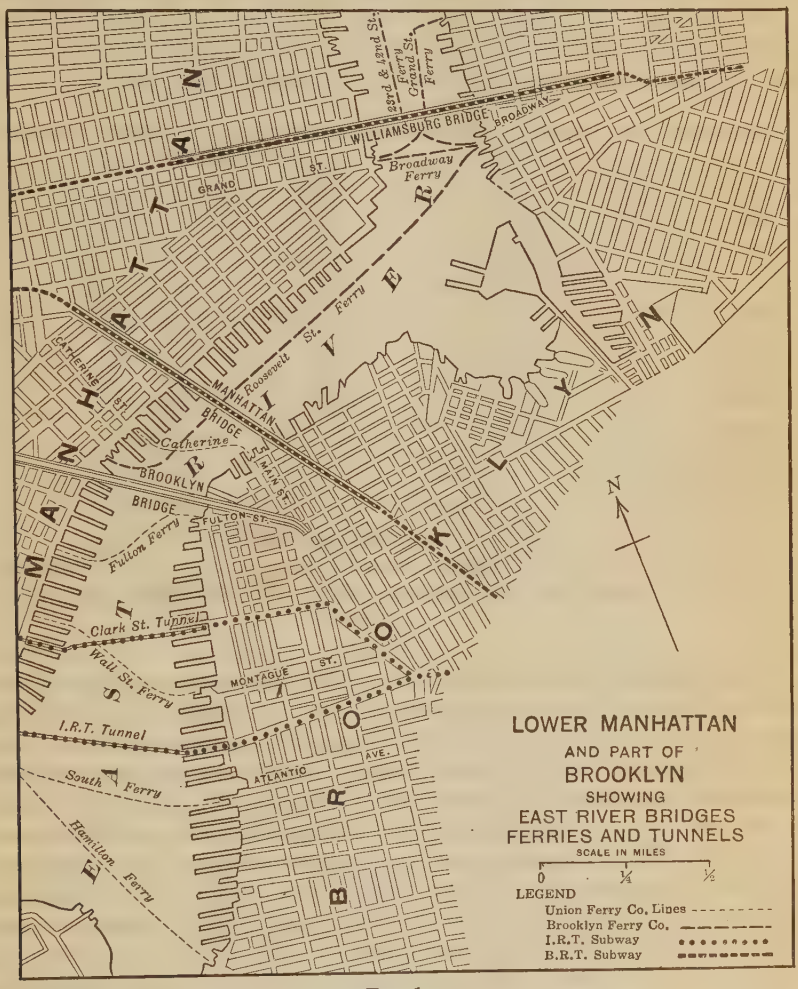

FIG. 1.

In 1886, the fare on the Bridge Railroad was reduced from 5 to 3 cents, and the trains were lengthened to 3 cars. This reduction made the differential between bridge and ferry rates during the greater part of the day only 2 cents on a round trip instead of 6 cents. Another factor affecting traffic in the following two years—1887 and 1888—was the general business depression. The

combination of these two influences resulted in a serious loss of business to the ferries. In 1889, the Union Ferry Company carried only 35 000 000 passengers, a loss of more than 30% from the high peak before the bridge was opened; the Bridge Railroad, on the contrary, continued to carry a heavier traffic each year, and, in 1890, actually exceeded the ferries in number of passengers, the figures being about 38 000 000 and 35 000 000, or a total of 75 000 000, including pedestrians, between Lower New York and Brooklyn. This represents an increase of 56% during the decade, as against an increase in the population of Brooklyn of only 40 per cent. It is evident, therefore, that after seven years of operation, the Brooklyn Bridge had not only robbed the ferries of a considerable part of their business, but had also created an absolutely new riding habit.

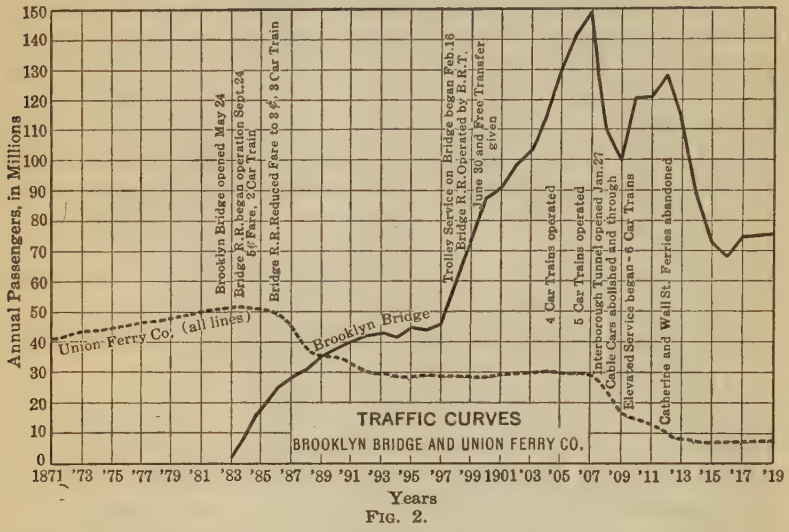

Fig. 2.

During the next ten years, the ferry traffic remained surprisingly uniform, being in 1900 only slightly less than in 1890. Meanwhile, the bridge traffic suffered a slight loss on account of the panic of 1893, but recoverd soon afterward. On February 16, 1898, trolley service was inaugurated, making possible a continuous trip from Park Row, New York, to various points in Brooklyn. In the same year, the operation of the Bridge Railroad was taken over by the Brooklyn Elevated System, and free transfers became effective between the two lines; this combination caused a tremendous jump in the bridge traffic, which practically doubled in the succeeding two years. Strangely enough, however, the ferries suffered no loss on this account; in fact, they carried more passengers in 1899 than in 1895 before the inception of trolley service on the bridge. A summary of the decade from 1890 to 1900 is almost identical with that of the preceding decade, as the population of Brooklyn again increased about 40%, while the total traffic across the river increased 59 per cent. From this, it appears that the improvement in the bridge facilities and the reduction in rates coincident with the opening of through service appealed strongly to the public and stimulated the riding habit.

No other important change in conditions occurred until 1908. Even the opening in 1903 of the Williamsburg Bridge, about 1¼ miles away, apparently had no effect whatever. The cable railroad across the Brooklyn Bridge successively increased the length of its trains, in 1905, to four cars, and, again, in 1907, to five cars. These changes were necessary to accommodate the constantly growing volume of traffic over the bridge, but the Union Ferry Company continued in much the same way as before, carrying about the same number of passengers as it had ever since the reduction of the bridge fare in 1886.

In 1908, however, the whole aspect of the situation was altered. In January of that year the first tunnels under the East River were opened; in the same year, the operation of cable cars on the Brooklyn Bridge was abandoned and through elevated service was inaugurated, using 6-car trains running directly to the New York Terminal. In spite of better transportation on the bridge, however, the traffic diminished sharply from more than 150 000 000 in 1907 to about 110 000 000 in 1908, due undoubtedly to the opening of the subway tunnels. This also was the determining factor in the utter ruin of the passenger business of the ferries. The traffic of the Union Ferry Company was running at approximately 30 000 000 per year in 1907. Then came the slump: 1908 showed a loss of 27% from the previous year and 1909 was 32% below 1908. As a result of the opening of the Manhattan Bridge in 1910, the Catherine Street and Wall Street lines were abandoned.

Vehicular Traffic.—Considering briefly the question of vehicular traffic, the bridges have never appealed strongly to horse-drawn vehicles. When the first bridge was built, and for many years thereafter, all vehicular traffic was horse-drawn. The high point of the roadway of Brooklyn Bridge is 135 ft. above mean high water, and in spite of the fact that the approaches run back to points elevated above the water-front, nevertheless, a hard climb is required of vehicles crossing the bridge. With the increase of automobile traffic, the importance of this feature has diminished, but there has always been sufficient horse-drawn traffic either originating near the docks, or preferring the easy ferry passage to the difficult bridge climb, to sustain the ferry service. It was this horse-drawn traffic that enabled the three remaining lines of the Union Ferry Company to continue until recently a moderately profitable operation.

Conclusions Drawn.—Consideration of the traffic over the Brooklyn Bridge and the East River ferries leads to several definite conclusions:

1.—As long as the bridge fare is considerably higher than the ferry fare, passenger traffic will not abandon the latter in favor of the former; this is shown by the fact that the Union Ferry Company lost practically no business while the bridge fare was 3 cents greater. When the difference was reduced to 1 cent, however, the ferries immediately lost 30% of their passenger business.

2.—Bridge shuttle service has relatively little appeal to the public. Traffic on the Brooklyn Bridge doubled in two years, after the inauguration of through trolley service from Park Row to points in Brooklyn. The increase in the two years from 1898 to 1900 was as great as during the previous fifteen years.

3.—A bridge with convenient transit facilities and a reasonably low fare will create a large volume of new traffic. This is proved inasmuch as the

total number riding between down-town Manhattan and Brooklyn increased more rapidly than the population of Brooklyn over the same period of years the increase being wholly on the bridge, as the ferry traffic either remained constant or actually decreased.

The Williamsburg Bridge

Prior to the opening of the Williamsburg Bridge, the traffic between Manhattan and the Williamsburg Section of Brooklyn was carried chiefly by the several lines of the Brooklyn and Manhattan Ferry Company, operated from Broadway, Brooklyn, to Roosevelt, Grand, East 23d, and East 42d Streets, Manhattan; from Grand Street, Brooklyn, to Grand Street, Manhattan; and from Greenpoint to East 10th and East 23d Streets, Manhattan. These routes provided the only transportation between the Williamsburg Section and that part of Manhattan directly across the river. The Brooklyn Bridge and the Union Ferry Company being 1¼ miles away, had no influence on the situation. A study of passenger traffic on the Brooklyn and Manhattan ferries substantiates the conclusions already reached in connection with the Union Ferry Company.

On December 19, 1903, the roadway of the Williamsburg Bridge was opened. Nearly a year later—November, 1904—trolley service was inaugurated (a local line across the bridge only); in February, 1905, through trolley operation was started; and in the fall of 1908, rapid transit service began. To quote from Vol. 2 of the Annual Report of the Public Service Commission for the First District of New York for 1916:

"The following are the most immediate effects * * * of the opening of * * * the Williamsburg Bridge in 1904: Discontinuance in 1908 of the following ferries: Grand Street, Manhattan, to Broadway and to Grand Street, Brooklyn; Roosevelt Street, 23d Street, and 42d Street, Manhattan, to Broadway, Brooklyn."

It should be noted that the ferry lines were abandoned in 1908, the year that subway service across the bridge was inaugurated and four years after the completion of the structure. Actual figures for individual ferries are not readily available. The two routes from Greenpoint, the traffic of which is included in the statistics, do not belong, perhaps, in the same category as the others in regard to bridge competition, but because the major part of the traffic was carried on the lines operating from Grand Street and from Broadway, Brooklyn, this inclusion does not alter the situation. In 1903, the Brooklyn and Manhattan Ferry Company was carrying 48 000 000 passengers per year, or nearly as many as the Union Ferry Company in the heyday of its prosperity. In 1904, the traffic remained constant, because no trolleys were used on the bridge until November. In 1906, after through trolley service had commenced, the ferry traffic decreased to 16 000 000, a loss of 67 per cent. The year that subway service began, the ferry traffic was 14 000 000, but the year following, it was only 9 000 000. These variations are shown in Fig. 3.

Thus, the history of the Brooklyn Bridge and the Union Ferry Company was repeated without any important variation when the Williamsburg Bridge was opened—the through service across the bridge was the factor that finally

spelled the doom of the ferries. It would appear that similar developments are likely to result from like conditions, wherever they occur. As an example, it may be interesting to consider the bridge now being built across the Delaware River from Philadelphia to Camden. This situation is so similar to the others that it should be possible to forecast the effect of the construction of the bridge on the local transit situation.

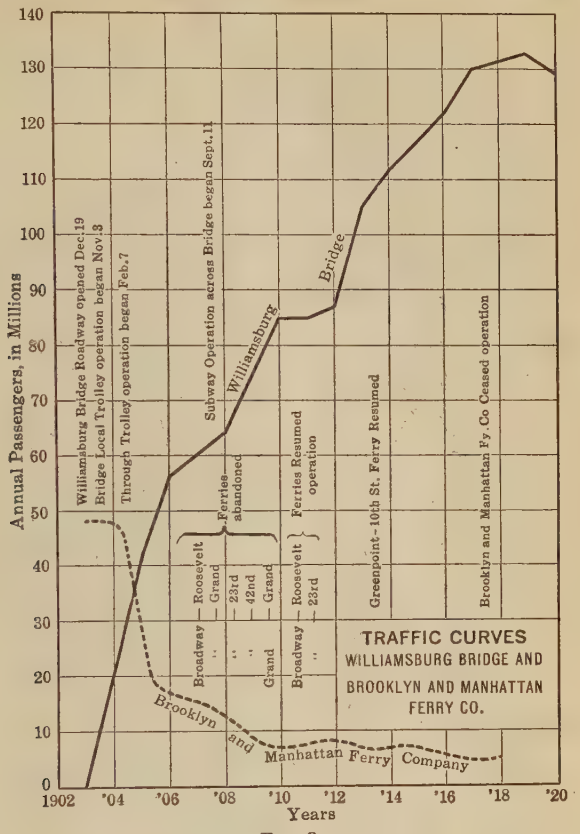

Fig. 3.

The Delaware River Bridge

At present, four ferry routes cross the Delaware River between Camden and Philadelphia (Fig. 4): (1) from Cooper's Point to Shackamaxon Street, Philadelphia; (2) from Cooper's Point to Vine Street, Philadelphia; (3) from Market Street, Camden, to Market Street, Philadelphia; and (4) from Kaighn Avenue, Camden, to the foot of Chestnut Street. According to the report of the Board of Engineers to the Delaware River Bridge Joint Commission, of June 9, 1921, passenger traffic is divided among these routes, as follows: Shackamaxon Street, 1%; Vine Street, 2%; Market Street, 77%; and Kaighn Avenue, 20 per cent. The first two routes are unimportant; likewise, the most southerly, Kaighn Avenue Ferry, need hardly enter into the calculations as its traffic is largely composed of passengers of the Philadelphia and Reading Rail-

road, who will patronize that ferry in any case, and a few others who use it for convenience alone. It has already been shown from the history of the Brooklyn and Williamsburg Bridges that traffic crossings a mile or more apart are virtually independent of each other. Hence, the use of the Kaighn Avenue Ferry is unlikely to be influenced by the new bridge.

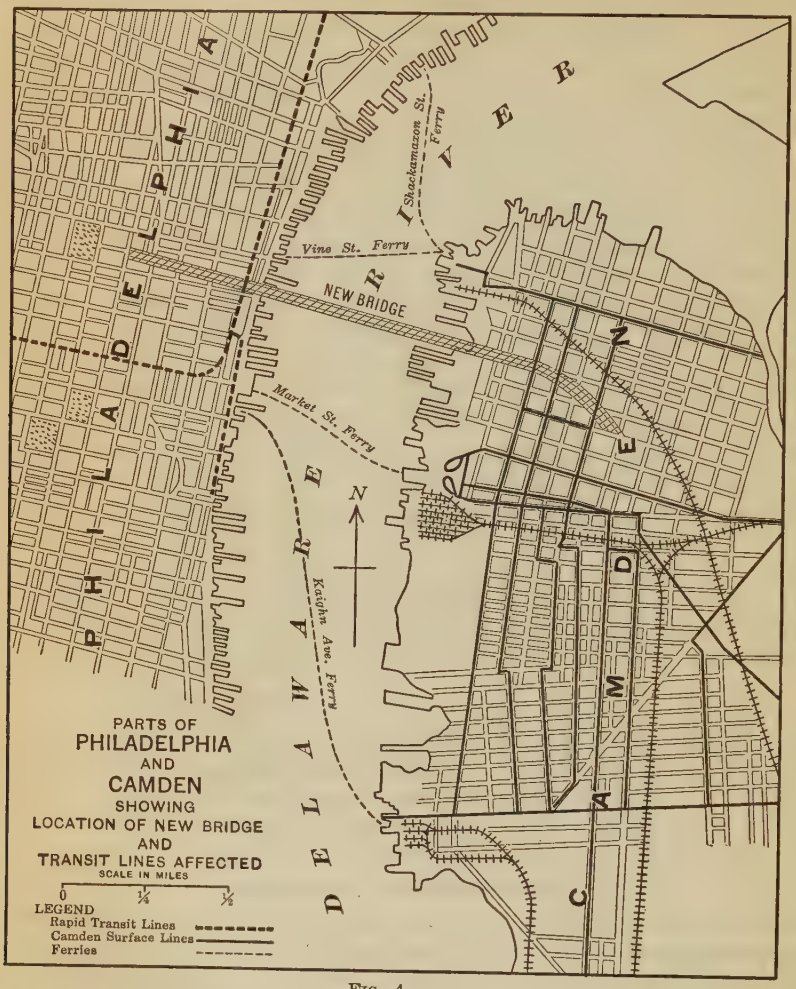

FIG. 4.

Ferry statistics for 1920 show that 37 000 000 passengers use the Market Street route, of which the Pennsylvania Railroad Company estimates that one-third uses the trains to and from the Camden Station. It appears unlikely that these travelers will be diverted to the new bridge as long as the Camden Station remains where it is. Moreover, the ferry is free to railroad passengers, and use of the bridge, therefore, would involve increased expense. Nor is it to be reasonably expected that the commuters from suburban points in New Jersey will desert the railroad in favor of other means of transportation, simply

because the latter crosses the bridge into Philadelphia. The corollary of the proposition that the Pennsylvania Railroad Company will retain its present traffic, is that Market Street Ferry will continue operation indefinitely, although possibly with less frequent service. A certain amount of horse-drawn traffic will probably continue to patronize the ferry, as in the case of the Union Ferry Company, because, at Philadelphia, many of the produce warehouses are located near the water-front.

One-third of the total traffic of this ferry consists of through railroad passengers who have no interest in local transportation in Camden and vicinity; the remaining two-thirds, numbering about 24 600 000 per year, or 67 500 per day, use trolley, or busses, or walk to near-by points. The effect of the new Delaware River Bridge on the local transit situation resolves itself into a question of what these daily local passengers will do. Referring to the history of the East River bridges in this connection, it is important to note some of the differences between the two situations. For instance, the ultimate destination of "Brooklynites" bound to Lower Manhattan is usually within easy walking distance of ferry and bridge terminals, whereas the ultimate destination of "Camdenites" bound to Philadelphia is apt to be some distance away from the ferry terminal at the foot of Market Street. Again, when the Brooklyn Bridge was built, the population of that Borough was several times as large as the present population of Camden.

In the report of the Board of Engineers to the Delaware River Bridge Joint Commission, previously mentioned, it is proposed that there shall be four tracks on the bridge, two for interurban rapid transit trains and two for a trolley shuttle-line. It is further suggested that there shall be constructed at some indefinite date in the future an elevated rapid transit system in New Jersey which shall use the bridge and the existing subway tracks of the Philadelphia Rapid Transit Company, thus connecting various suburban points directly with the heart of Philadelphia. As an ultimate solution of the transportation question in this area, the plan is not without attractive features, but as a possibility for the near future it is hardly thinkable—the cost of construction would be out of all proportion to the present traffic demands of Camden and its environs.

There are, however, several ways of utilizing the tracks, without involving expensive new elevated construction work:

1.—A shuttle-line, as proposed in the report to the Joint Commission, previously mentioned, crossing the bridge and looping in Camden *via* Fourth, Mickle, and Fifth Streets. Existing transportation agencies would continue operation substantially as at present.

2.—Operation across the bridge of either the Philadelphia subway or surface lines to a terminus at the Camden end.

3.—Operation of the Camden lines across the bridge to a terminus at the Philadelphia end.

4.—Some combination of the foregoing plans.

Shuttle Service.—In case shuttle service is operated, the length of the route will be about 4 miles per round trip. A fairly short headway would always be

necessary, because people are used to frequent ferry service; an even shorter headway would be required to take care of the sharp rush-hour peaks. On a 4-min. schedule, this would mean at least 300 trips per day, or 1 200 car-miles. The annual operating expense, at 40 cents per car-mile, would be $168 000.

It is difficult to estimate the exact number of passengers who would prefer such service to the present ferries. Although the shuttle had little effect on the East River ferries, nevertheless the fact that the bridge terminal is more centrally located than the ferry terminal in Philadelphia might make such service slightly more attractive to "Camdenites" than it proved to be to Brooklynites". New Jersey people desiring to reach a point within easy walking distance of the Pennsylvania end of the bridge would probably save money by paying one fare on the bridge shuttle instead of paying 4 cents on the ferry and 7 cents in Philadelphia. On the other hand, those whose destination is more or less remote from the bridge terminal, would still have to take local transportation in Philadelphia, and unless the bridge fare was less than 5 cents, they would be paying more money by using the shuttle. Moreover, for a person who uses local transportation in both cities, a short ferry ride is a pleasant interlude, and the substitution of another rail trip for his present water trip would have little appeal.

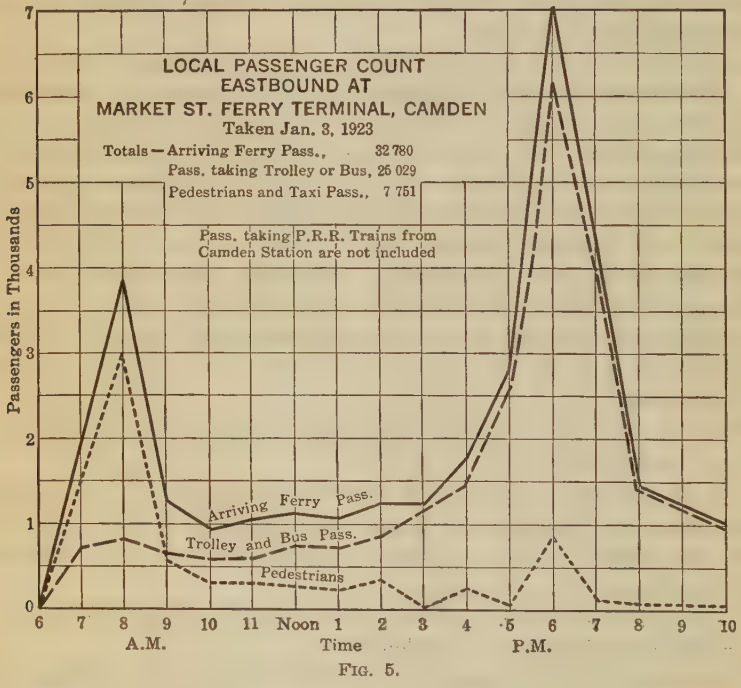

FIG. 5.

The assumption that the traffic on a shuttle-line would approximate the number of people who now walk to and from the ferries in Camden, would probably not be far wrong. From Fig. 5, the total number of pedestrians (both directions) is shown to be about 15 000, of which number, 6 000 are carried by

the ferries in the two rush hours—eastward in the morning and westward at night. On a basis of 100 passengers per car, the 4-min. headway of the shuttle cars would accommodate 1 500 per hour as a maximum, and thus reduce the possible total passengers to 12 000 per day. The present local fare on the Williamsburg Bridge is 1⅜ cents, on the Brooklyn Bridge, 2½ cents, and on the Market Street Ferry, 4 cents. In view of the conclusions reached by a study of the traffic on those two bridges, the maximum fare possible on the Delaware River Bridge shuttle appears to be 3 cents. The annual revenue would then be $126 000, or $42 000 less than the estimated costs. It would be impossible to justify a larger estimate of traffic on a bridge shuttle-line on the ground of experience elsewhere. Under these circumstances, the plan can be dismissed as impractical, because it would be neither popular or profitable.

Extension of Philadelphia Lines.—The next possibility to be considered is the operation of the subway trains of the Philadelphia Rapid Transit Company across the bridge to a terminus in Camden. This seems only slightly less improbable than the plan already discussed. The bridge terminal in Philadelphia will be so far back from the river and so far north of the present subway under Market Street, that it would be no easy matter to get the trains to the bridge. Moreover, in so doing, subway service to the lower end of Market Street and to the ferries would be sacrificed. Much additional mileage would be added to the operation if Camden was made the terminus in place of the Philadelphia water-front. It would hardly be feasible to split the service, sending some across the new bridge and some to the ferries, because it is already split between Frankford and the ferries. As the ultimate destination in Philadelphia of ferry passengers is probably the vicinity of the City Hall, the majority of them now use the Philadelphia Rapid Transit lines. There would be no increase in traffic to this Company, therefore, if it were to operate over the bridge. Unless an extra fare across the bridge should be charged, there would be no increase in revenue to counterbalance the added mileage and the traffic lost by failure to serve Lower Market Street.

Similar criticisms can be made of projects to send Philadelphia trolleys across to Camden. Although the latter could be arranged with less construction cost, the same loss of traffic on parts of their present routes and the same increased mileage without increased revenue would result. From every point of view, it is undesirable to tie up the Philadelphia transportation system—the primary mission of which is to serve that city—with bridge service of an entirely different nature.

Use of Bridge by Camden Trolleys.—Operation over the Delaware River Bridge from Camden to Philadelphia, on the other hand, is a more attractive plan. Of the 65 000 daily local passengers on the Market Street Ferry, 76% now use the trolleys or busses in Camden and approximately 24% walk (Fig. 5). With cars running through from the New Jersey side directly into Philadelphia, a large proportion of the pedestrians probably would become passengers, because the inevitable curtailment of ferry service would make the bridge the natural route for crossing the river. Although not all of them would neces-

sarily ride across the bridge, nevertheless, it has been the experience on the Brooklyn and Williamsburg Bridges that the number of pedestrians is inconsequential compared to the rail passengers, averaging less than 3% of the total. Assuming that one-half the people who now walk to the ferry, would take transportation across the bridge if no extra fare was charged, this would give a traffic increase of 8 000 per day, or 2 800 000 per year. It will be seen that this figure is smaller than the traffic estimate already given for a bridge shuttle-line. This is natural if the existing transportation agency charges its regular fare, which is higher than that which could reasonably be asked on a shuttle-line. Hence, fewer pedestrians would become riders. To counterbalance that, however, is the fact that it would cost much less for the present lines to extend their service across the new bridge, than for a shuttle-line to operate thereon. The volume of travel between Philadelphia and Camden has been increasing steadily, as shown in Fig. 6, and with the completion of the bridge may be expected to increase even more rapidly. This increase in the riding habit would soon greatly augment the number of passengers, and even after allowing for rather wide variation in these figures, the plan remains a promising one.

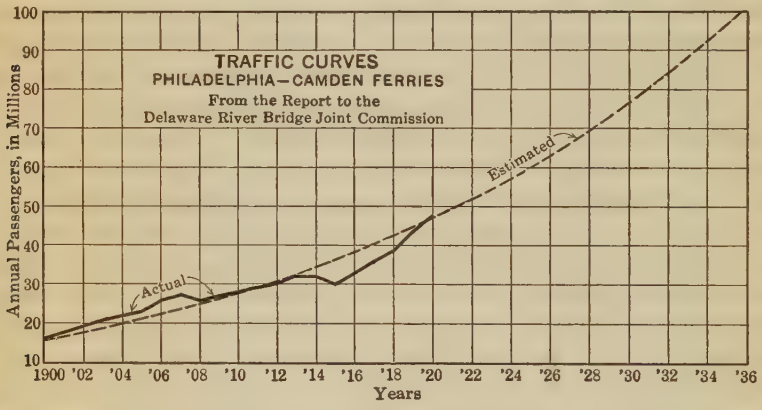

FIG. 6.

Possible Combined Plans.—The fourth alternative is a combination of two of the plans already mentioned. It is evident that the shuttle service would be unprofitable even though operated as a monopoly on the bridge; if operated alongside of through service, it would be even more unprofitable. The disadvantages incident to the operation of the Philadelphia lines by themselves across the bridge weigh equally against any combination plan.

The fact that operation across the Delaware River Bridge would involve interstate transportation is an important consideration. Probably neither railway is desirous of coming under the jurisdiction of the Interstate Commerce Commission, and a profitable business would have to be assured before either company would undertake such operation. Then, too, the present population of Camden and its environs is much smaller than that of Brooklyn in 1883, so that no such volume of traffic as that carried by the Brooklyn Bridge in

its early days could be expected for many years. On the other hand, one of the existing transportation agencies, rather than see its territory invaded by another, might prefer to give a service even at a sacrifice, in order to keep out competitors.

Comparison of Plans.—In view of the history of the East River bridges, the most logical program appears to be, therefore, that the local transportation lines in New Jersey extend their service across the new Delaware River Bridge to Philadelphia. Such a development would be in harmony with the true mission of these lines, which is to bring New Jersey suburbanites into the city. Moreover, it presents no serious construction problems, and is economically the most promising of the four plans.

Conclusion

Similar bridges are being considered elsewhere, as, for example, from New York, across the Hudson River, to New Jersey. In such cases, the general conclusions reached from a study of the East River bridges would probably be applicable, and should receive careful consideration before the adoption of final plans for the rail service involved.

DISCUSSION

CHARLES EVAN FOWLER,* M. AM. SOC. C. E.—The author has presented only that feature of a traffic bridge which is of importance to those engaged in rapid transit, namely, its use for passengers; the vehicular traffic, which is of paramount interest to the builder of a toll bridge, has scarcely been mentioned. The income from passengers crossing a toll bridge will represent only about 5% of the total revenue of a bridge some distance from a city; it may amount to 40% for an inter-city structure that carries rapid transit. The major part of this income must come from automobiles, auto-trucks, and auto-truck freight. Such traffic was not available in sufficient volume even a few years ago to make possible the financing of the many much needed bridges.

The speaker's detailed study within the last few years of ten or twelve large projects of this type throughout the country, and his close observation of fifty or more toll bridges now in operation, have convinced him that except in extreme cases the passenger traffic from the income standpoint is either negligible or must be subordinated in the design to vehicular traffic. Moreover, automobiles will use a bridge to the exclusion of a ferry, when the bridge entrance is within 2 miles of the ferry crossing, and in case the traffic is of average density; but if the traffic is light, the distance may be much more than 2 miles and still the bridge will be used in preference. This is due to the fact that an automobile needs on an average at least 15 min. merely to get on the boat, but during rush hours, and on Sundays and holidays, this time may be from 1 hour to several hours.

The bridge over the Columbia River between Portland, Ore., and Vancouver, Wash., may be taken as an example of the class of bridge situated some distance from a city. This crossing was served by one small ferry until February, 1917, when a bridge was completed at a cost of considerably less than $2 000 000. The ferry simply fell into disuse; and the traffic has increased to such an extent that the net earnings of the bridge will exceed its cost in about six years. The curves established for passenger and automobile traffic over the Philadelphia-Camden Bridge, as shown in the report of the Delaware River Bridge Commission, indicate a probable traffic in 1927 of more than 60 000 000 passenger crossings, and more than 4 000 000 automobile crossings. Due to the probable increase in population and in the number of automobiles, the speaker would place this latter expectation at 6 000 000. Also, there will be, in all probability, upward of 1 500 000 crossings of auto-trucks, carrying about 6 000 000 tons of freight. If this structure should be operated as a toll bridge, the income for the first year would surely exceed $5 000 000.

It is often the case with engineers and generally the case with public officials that they do not fully grasp the trend of modern vehicular traffic. There were about 15 000 000 gasoline vehicles in use in the United States in 1923; with more than 4 000 000 machines being manufactured each year, the number may be expected to reach more than 25 000 000 by 1927, even after deducting those scrapped and otherwise destroyed. It is fair to assume that each summer

* Cons. Civ. Engr., New York, N. Y.

will find 60% of these automobiles on tour, or about 15 000 000 by the summer of 1927 to complicate the bridge-traffic problem. The increase in the number of automobiles in large cities is much more rapid—about 30 to 40% per annum. The increase due to this summer traffic each year will be about 100% above the normal local annual traffic, and about 50% above the average. This, however, does not represent fully the problem for those who are planning large bridges, as the daily peak load each week will be more than 50% greater than the average daily traffic, and the hourly peak loads, which are not absolutely prohibitive now, will become so in 3 or 4 years. By "prohibitive" is meant that no bridge can be made to pay if constructed to provide for anything approximating the peak loads that will surely come within 2 or 3 years.

The speaker's studies for a long-span bridge across the Detroit River between Detroit, Mich., and Windsor, Ont., Canada, were based on the traffic of the trunk highways and of the city streets, but particularly on extended tallies of the present ferry traffic. This would indicate not less than 15 passenger crossings per capita per annum, as compared with 30 between Philadelphia and Camden, and 50 between San Francisco and Oakland, Calif. The population for Detroit, Windsor, and their environs was estimated as being 1 650 000; for Philadelphia, Camden, etc., as 2 000 000; and for San Francisco, Oakland, etc., as 1 000 000. On the basis given, and in consideration of other data gathered, the minimum traffic for the Detroit River Bridge for 1927 would be as follows:

Passengers	25 000 000
Automobiles	3 000 000
Auto-trucks	700 000
Auto-truck freight, in tons	3 000 000

The maximum traffic would be:

Passengers	50 000 000
Automobiles	5 700 000
Auto-trucks	2 200 000
Auto-truck freight, in tons	8 800 000

The corresponding gross income would range from a minimum of about $2 500 000 to a maximum of more than $6 000 000, the larger estimate being tenable on account of the very large traffic to and from the 200 Canadian branches of American manufacturing concerns.

The Detroit Bridge is also planned to carry two rapid transit tracks, but in the speaker's opinion all such traffic for New York should be carried by tunnels with electric operation, and the bridges constructed so as to care for all vehicular traffic, including probably auto-busses.

The possibility of traffic tunnels proving moderately successful does not alter the facts in the case of tunnel *versus* bridge, as there are enough physical, physiological, and psychological reasons against tunnels for automobile traffic to make it unnecessary to calculate relative costs. Tunnels are best suited for handling electric rapid transit and bridges, for automobile traffic. Thus, it is easy to see the logical solution of the problem in New York—to construct additional tunnel connections for the subways across the Hudson and

East Rivers, so as to remove all tracks from the East River bridges, which will then have enough additional carrying capacity to care for automobile traffic for some years to come; then to construct two bridges over the Hudson River for automobile traffic alone, and three double tube tunnels for rapid transit. That a tri-borough automobile bridge over the East River will be needed eventually must be admitted. If the rapid transit tracks are removed from the old Brooklyn Bridge, it can be remodeled at a cost of less than $15 000 000 to care for a large portion of the down-town automobile traffic.

The construction of facilities on this general scheme will solve the problem of river crossings in a most economical and satisfactory manner for other cities. Most of the large cities in this country are suffering from a plethora of ideas and of city commissions. The creation of a General Utilities Commission, with departments to handle each particular civic problem, would make possible a real co-ordination of all the public utilities of a city, and also make it possible to carry out the necessary construction in a logical and progressive manner, so as to keep pace with the public need for the most urgent improvements.

M. W. Weir,* M. Am. Soc. C. E.—Most people will do what is easy, convenient, and time-saving, both from the standpoint of physical and financial effort.

A bridge becomes an important link in the transit systems of the places it connects. Its importance lies in the fact that it has provided the means of co-ordinating the transit systems and traffic lanes on both sides of the physical barrier which it overcomes, and its full effect on the population of the satellite city is not manifest until this co-ordination is so far completed as to make it both physically and financially easy, convenient, and time-saving for the public to use the bridge.

This paper, in noting the discrepancy between the increase in population of Brooklyn and the increase in passenger traffic across the river, registers surprise; but this relation is accounted for by the fact that practically all the people that took up residence in Brooklyn, became potential passengers—in fact in going to New York and back each individual became in the records two passengers—and to these must be added the traffic due to business created by the increased population on both sides of the river.

It is interesting to note in Fig. 2 that the bridge traffic has a general recession from 1907 when the Interborough Tunnel was opened, which may show the influence of still another element that only began to demonstrate itself materially at about this time—the automobile. The frantic struggles of trolley companies to keep alive during the past few years, even with increased fares and more or less neglected maintenance, cannot be laid to increased population, but must be accounted for by the increasing use by the public of the automobile and the bus. In all these cases the new method has succeeded because it is easy, convenient, time-saving, and financially economical.

Conclusions drawn from past performances where all the conditions bearing on the question are fairly constant, may be relied on to a large extent in fore-

* Cons. Engr., New York, N. Y.

casting a future performance, but to draw conclusions from the past performance of the transit over Brooklyn Bridge, or any of the other East River bridges, either singly or collectively, is apt to be misleading, unless such a forecast pertained to a bridge surrounded by similar conditions; and while in another place the physical conditions may be the same, the habits and temperament of the people and their occupations may be so different as effectually to upset a calculation based on one comparison.

In comparing the traffic to be expected and the method of handling it over the Philadelphia-Camden Bridge, the physical conditions on the Pennsylvania side of the river are immeasurably different from those on Manhattan Island. There is no barrier like the Hudson River to shut off the expansion of Philadelphia to the west, no barrier like New York Bay to shut off expansion in a southerly direction. In fact, Philadelphia is free to expand in every direction except eastward.

For unexplained reasons cities show a decided tendency to expand toward the west when, as at Philadelphia, no barrier exists to thwart this trend. With ample room to grow, it only remains for rapid transit of one type or another to stimulate that growth, for it is manifest that city people no longer calculate distance in terms of miles but in terms of minutes.

This physical difference between Philadelphia and Manhattan is sufficient in itself to upset a forecast based on past performances of Manhattan-Brooklyn Bridges. In addition there is the further difference that Manhattan is a great harbor port and selling center where comparatively no large manufacturing plants are operating, while Philadelphia is a manufacturing city located many miles from the sea.

Brooklyn has been termed the "city of homes, rubber plants, and baby carriages," while Camden is an industrial center; furthermore, Brooklyn is a seaport of importance, while Camden is a river port many miles from the sea. The occupations of the people correspond to the difference between a manufacturing center and a selling center.

The effect of the Philadelphia-Camden Bridge on the vehicular traffic over the Market Street and Vine Street ferries will probably be noticeable in so far as light vehicles are concerned, particularly on Sundays and holidays when the traffic to Atlantic City is heavy. Truck traffic originating along the river front will probably continue to use the ferries to a considerable extent for economy's sake.

Passengers to trains in Camden, except those originating within a short distance of the Philadelphia end of the bridge, will find but little advantage in the bridge. Passenger traffic in either direction over the bridge would become of moment only if continuous transportation to the environs of the two cities is accomplished with an expedition, convenience, and economy, at least equal to that of the channels now in operation.

Of the nearly 4 000 passengers arriving in Camden from Philadelphia in the morning (Fig. 5), 3 000 were pedestrians, largely workers in the adjacent Camden factories; the same chart shows that, in the evening, of the 7 000 passengers, less than 1 000 walked from the ferry. This indicates that more

than 6 000 lived at a distance from the ferry; personal observation would lead to the belief that the number traveling to the suburban villages just outside of Camden was large.

Shuttle service across the bridge would be of doubtful advantage to the public and of precarious existence financially as it would have such a limited area for supplying passengers. This is believed to be true of all the bridges.

It is doubtful if either the trolley companies of New Jersey or those of Pennsylvania will be at all interested in crossing the Delaware River Bridge. Trolley companies all over the eastern part of the country must feel that their existence is seriously threatened. The recent strike of trolley employees in New Jersey demonstrated that motor-bus transportation, properly developed, would have decided advantages; for instance, the busses do not occupy fixed positions in the street, they can pick up and deliver passengers at the curb, and they do not present an obstacle to street traffic as does the trolley car. The capacity of the street is vastly increased by eliminating the trolley car—a matter of considerable importance to cities with narrow streets—and the busses are rapid and not subject to delays originating in preceding vehicles. Furthermore, the attitude of the public is also a strong argument against the trolleys. Taking all these things into consideration, it would be a doubtful venture for trolleys to undertake a program of extensive betterments and extensions, particularly so when it is considered that crossing a bridge like the Delaware River Bridge, entails construction and maintenance for a considerable distance without creating any additional points for passenger collection. As a business proposition it is not enticing.

Operation over the bridge of a transit line which reaches not only into Camden, but well beyond into the suburbs, might induce Philadelphians to live in New Jersey provided they could do so conveniently and more cheaply than in Pennsylvania, particularly if, after reaching Philadelphia, they could continue to their destination without change. Such an arrangement entails, of course, all the difficulties of interlocking corporations, political boundaries, and human elements, pugnacious and irritating. That it is not impossible to attain such perfection of transit is exemplified by the subway systems operating in Manhattan and Brooklyn.

At this time surface transit systems are so threatened by more modern methods that it is doubtful if former accomplishments can be used as a basis for speculation.

The Delaware River Bridge is best fitted to serve its primary purpose, namely, to accommodate vehicular transportation, of which the Sunday and holiday traffic from Philadelphia to New York and from Philadelphia to Atlantic City will be the outstanding feature. It will be Pennsylvania's gateway to the sea.

Herman H. Smith,* M. Am. Soc. C. E.—The study of the tremendous increase in the traffic between Brooklyn and Manhattan is interesting and involves consideration of certain changes in the economic life of the two com-

* Deputy Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

munities which, aside from the growth in population, has had a direct effect on this volume of interborough traffic.

The author has pointed out that during certain periods the traffic across the East River has increased, relatively, much faster than the population of Brooklyn; and from this fact he draws the conclusion that the reduced cost and increased convenience of transportation over the bridges created a large volume of new traffic. While unquestionably the best and cheapest service will attract the most traffic, conditions in New York are such that great care must be exercised in drawing conclusions of a general character from the results here obtained, if it is intended to apply such conclusions elsewhere.

The Island of Manhattan is the great business, commercial and industrial, center of the City of New York and its environs. The island's commercial growth has curtailed its residential growth and many former residents of Manhattan now help to swell the tide of daily traffic which surges back and forth between it and Brooklyn.

A tremendous business and industrial expansion is taking place continuously on Manhattan Island; it is quite evident that this growth requires each year more and more workers, whom Manhattan cannot supply, and who, therefore, must come from the surrounding territory. These conditions have caused an abnormal increase in the population of the adjacent communities, as compared with Manhattan itself. It is this great outside population, a large percentage of which is dependent on the business and industry of the metropolis which supplies the bulk of workers for Manhattan's commercial activities. Of all the other Boroughs, Brooklyn lies nearest, with a population about equal to that of Manhattan, so that here exists a condition probably unequalled as an inter-community traffic problem.

Another economical condition has also entered into the traffic problem and has helped to increase this interborough traffic at a much greater rate than the increase in population alone would indicate. At the time the Brooklyn Bridge was opened, about forty years ago, comparatively few women who lived in Brooklyn were engaged in business or industry in Manhattan. Subsequently, each year has seen more and more women thus employed, so that now a much larger percentage of the population of Brooklyn is traveling back and forth each day between the two boroughs. Aside, therefore, from the fact that better and cheaper transportation has tended to create more traffic, this change in the economic condition has probably been the main factor in increasing the traffic at a faster rate than the population.

Whether or not the lessons which New York has taught with respect to inter-community traffic can be applied to good advantage in connection with the construction of bridges in other communities, will depend on local conditions and the judgment used in applying the results. Manhattan is an island surrounded by a population three or four times as great as its own population. The population of Manhattan cannot grow to any appreciable extent, whereas its business and industry are expanding at a rate which makes it necessary to draw more and more on the surrounding territory for its workers. This condition is probably unparalleled in any other community; consequently, it is easy to be misled by the traffic growth which has resulted from this unique situation.

D. B. STEINMAN,* M. AM. SOC. C. E.—The analysis of the traffic development on the Brooklyn and Williamsburg Bridges, presented by the author, is instructive and interesting, and his conclusions appear to be logical deductions from the data submitted. His principal conclusion, to the effect that rail facilities on a bridge are best utilized as the extension of the transit lines in the minor of the two communities connected by the structure, will hardly be disputed.

It is to be regretted that the author has focussed attention chiefly on what is properly the secondary division of traffic on highway bridges, namely, the rail traffic. It would be extremely desirable if he could present similar traffic curves and conclusions in regard to the major division of highway bridge traffic, the vehicular or automobile traffic.

In connection with the Brooklyn Bridge, his traffic curves show a development of the volume of passenger traffic to the amount of 300% of the maximum passenger traffic on the ferries prior to the construction of the bridge. In the case of the Williamsburg Bridge, the volume reached was 270% of the previous maximum ferry traffic. It would be desirable if, in his closing discussion, Mr. Miller could give similar relations for the vehicular traffic on the bridges as compared with the previous vehicular traffic on the ferries.

The author's arguments have been principally directed to the question of the best use that can be made, after the completion of the bridge, of rail facilities provided on the structure. A somewhat different angle of the same problem is the question of the conditions under which it is justifiable or advisable to provide rail traffic facilities on a highway bridge.

In connection with recent studies for various bridge projects, the speaker has come to certain conclusions on this question, and the facts and figures presented in this paper seem to bear out those conclusions. In his discussion of the best use of the tracks on the Delaware River Bridge, the author has shown that their utilization for shuttle service would be neither popular nor profitable; he has shown that the use of the tracks as an extension of transit lines now existing in Philadelphia would result in a loss of revenue to the operating companies; and he has suggested, as the most promising solution of the problem, the utilization of those tracks for extensions of present transit lines on the Camden side. Even with the last solution, however, he does not promise any positive profit to the operating company.

This questionable showing as to profit for the operation of the tracks on the bridge emphasizes the importance of the investment represented by these rail traffic facilities. Of the total live load calculated for the Delaware River Bridge, 1 000 lb. per lin. ft. represents foot traffic, 4 000 lb. per lin. ft. represents wheel or vehicular traffic, and 7 000 lb. per lin. ft. represents rail traffic. Of a total of 12 000 lb., 7 000 lb. represents the provision for the traffic on the four tracks. In other words, about 60% of the live load capacity of the bridge is provided to take care of its anticipated rail traffic. It would appear, therefore, that if the tracks had been omitted from the structure the total cost might have been reduced 40 or perhaps as much as 50 per cent.

This investment in the initial cost of the structure should be considered in arriving at any judgment as to the economic justification of providing these

* Cons. Engr., New York, N. Y.

tracks on the Delaware River Bridge. It is a question whether any company would undertake to operate those tracks if, in addition to its ordinary operating expenses, it would have to pay interest on the investment represented by the track provision in the first cost of the structure.

Similar conclusions can be drawn from studies of other bridges. There are a number of toll bridges crossing the Ohio River, which provide for both wheel and rail traffic. The speaker recently had occasion to go over the figures of earnings and expenses on those bridges, and was surprised to find that of the gross earnings only 20 to 30% was produced by the rail traffic; and this, in spite of the fact that the rail traffic represented the larger proportion of the first cost of the structures, also the greater proportion of the annual charges for maintenance, repairs, and depreciation.

These facts certainly place a question mark against the economic advisability of providing for rail traffic on highway bridges. Highways are used primarily for vehicular traffic, and a highway bridge is merely an extension of the highways, a connecting link to close an existing gap; and it would appear logical, therefore, to use the bridges for the same kind of traffic that dominates the highways of which the bridge is a continuation.

The problem may be somewhat different in the case of a large city bridge, but even in that case similar conclusions are justified. In a city bridge, it may be desired to provide for continuation of the trolley or rapid transit service found in the city streets. Trolley service, however, is rapidly being superseded by the more modern system of bus service, which appears to be the best solution under conditions of growing street congestion. Rapid transit service in cities is either in subways or on elevated tracks. Elevated construction is coming to be regarded as obsolete; the City of New York now contemplates going to a great expenditure of money to eliminate the existing elevated structures as obstructions to light and traffic. For the rapid transit lines carried in subways, it would appear more logical to carry the traffic across the river in tubes. Accordingly, in cities, as well as in rural districts, there appears to be little economic justification for placing rails on a highway bridge.

There is another phase of this question: In promoting a bridge project, the most difficult problem is not that of engineering, but of financing, and any overloading of the financial end of the problem makes it more difficult to realize the project; that fact should be kept in mind before burdening the project with an element of questionable economic justification.

In conclusion, the speaker submits that a highway bridge should be built primarily for the natural traffic of the highways, which is vehicular traffic, and that the increased investment required for rail traffic provisions will generally fail to contribute a commensurate increase in usefulness or revenue-producing value to the structure.

H. M. Lewis,* M. Am. Soc. C. E.—The paper refers only to bridges of large span connecting communities each of which have their own business and residential centers. Limiting his discussion to this type of bridge, the speaker believes that, in general, trolley lines should be extended across the bridge from the smaller community to a terminal in the larger one. Heavy trains, such as

* Executive Engr., Regional Plan of New York and Its Environs, New York, N. Y.

are carried on elevated railroads or in a subway system, should be extended at least into the business centers of both communities. The minimum facility desirable at either end of the bridge would be a loop connecting, if possible, with another parallel bridge, distributing and collecting the passengers to and from several points. A terminal operation of bridge trains in a large city is apt to lead to such congestion as occurred at the New York end of the Brooklyn Bridge, and which has not been entirely relieved even by the addition of several other routes leading from Brooklyn to Southern Manhattan.

Bridges of large roadway capacity are apt to create serious street-traffic congestion at their terminals, particularly as the street system, generally laid out long before the bridge was planned, is almost certain to be ill adapted for such approaches. The farther rail passengers can be discharged from this point of street congestion, the better it will be for general conditions.

The effect of bridges on transit conditions depends not only on the location of the bridge in reference to existing cross-river facilities, but may be vitally influenced by the following factors:

1.—The topography of the shores and their adaptability to the economical construction of transit connections.

2.—The type of transit system, existing or proposed, in the communities which the bridge connects, that is, whether it is a surface, elevated, or subway system.

If there is low land on both shores, the provision of approaches for rapid transit lines becomes an expensive item, but trolley connections and terminals can be installed much more easily. Where the shores are relatively high, connections with existing rapid transit lines, either elevated or subway, is greatly simplified. The ease with which such connections can be made thus becomes important in determining the feasibility of carrying transit facilities across a bridge.

In New York, the East River bridges will undoubtedly have less and less effect on transit conditions, as the subway tunnels will become more and more the main passenger carriers. If it is ever possible to abolish the elevated railroads in down-town Brooklyn (surely desirable and quite possible in the next generation), the Brooklyn Bridge will become still more negligible as a transit factor. The Manhattan Bridge, due to its strategic location and great capacity, has largely superseded the Brooklyn Bridge as a transit carrier, while the Williamsburg Bridge, directly connecting the most densely populated parts of Brooklyn and Manhattan, carries far more transit passengers than any of the other East River bridges. Counts made by the Department of Plant and Structures, in November, 1922, indicated that the total passengers carried annually by the trolleys and the elevated railroad and subway cars on the East River bridges were distributed about as follows:

Williamsburg Bridge	149 000 000
Manhattan Bridge	96 000 000
Brooklyn Bridge	49 000 000
Queensborough Bridge	25 000 000
Total	319 000 000

It surely is hardly fair to classify bridges that carry such a large number of passengers on transit lines primarily as highway bridges.

There is a rapid transit tunnel now almost directly beneath the Queensborough Bridge and there will undoubtedly be future tunnels very close to the other bridges.

Because of the high shores on the New Jersey side, the Hudson River Bridge, if built, would be especially adaptable for transit facilities. If this bridge were built at a point in Upper Manhattan, such as Fort Washington Park, where the New York shores are also high, connections on both sides would be relatively simple. Transit facilities at this point, however, should form part of a circumferential transit line, as it is too far north to serve as one of the principal arterial connections to Lower Manhattan, and the use of it in this connection would throw too great a burden on existing or proposed north and south routes within the city. If facilities could be provided for bypassing the congested parts of Manhattan and connecting Northern New Jersey with the apartment section of Manhattan and the Boroughs of the Bronx and Queens, the increased facilities for connection between these outlying parts of the city would not only save long delays for persons now wishing to pass between those points, but would facilitate a decentralization of business activities.

In response to Mr. Fowler's question as to the total figures for subway traffic between Manhattan and Brooklyn, it may be of interest to note that the Engineering Division of the Regional Plan of New York and Its Environs recently compiled some figures showing the distribution of passenger traffic between Manhattan—south of 59th Street—and the surrounding areas. These indicated that the total number of persons that would probably enter (one-way traffic) this district during 24 hours on a typical business day in 1924 would be distributed about as follows:

Brooklyn-Manhattan Transit and Interborough Rapid Transit Tunnels to Brooklyn	318 000	
Brooklyn-Manhattan Transit and Interborough Rapid Transit Tunnels to Queens	102 500	
Pennsylvania and Long Island Railroad Tunnels to Queens	48 500	
Total		469 000
By the four East River Bridges, including passengers by rapid transit, trolleys and vehicles, and pedestrians		628 000
By ferries on the East River, south of 59th Street		19 900

Assuming that a typical business day is one-three hundred and fortieth of the annual total, the corresponding 1924 totals for traffic both in and out would be 319 000 000 by tunnels; 427 000 000 by bridges; and 13 600 000 by ferries. Thus, it is seen that of the total passenger traffic between Long Island and Manhattan—south of 59th Street—56% is carried by bridges, 42% by tunnels, and only 2% by ferries.

T. Kennard Thomson,* M. Am. Soc. C. E.—The increase in passenger traffic from Brooklyn since 1871 (Fig. 2) is a reminder of the interesting paper† by F. W. Gardiner, M. Am. Soc. C. E., in which it was stated that the Elevated Railroad Company had advertised, in 1872, that it was carrying 1 300 passengers per day and had solved the transit problem for New York. This compares with the 2 500 000 passengers now carried daily on all transit facilities combined.

About twenty-six years ago, the speaker published an article in *The Brooklyn Eagle,* in which he stated that the only way to solve the terminal problem in New York was to abolish the terminal.‡

To build a bridge solely for highway traffic, as suggested by Mr. Steinman, either in New York or in Philadelphia, would surely be to serve the automobile owning class at the expense of the masses.

One question which has not been considered in the discussion, but which has been fully covered in the *Transactions* of the Society, is the damage to property along the water-front, due to the construction of the East River bridges. It will be interesting to note whether a similar result occurs in Philadelphia and Camden.

Herbert C. Keith,* M. Am. Soc. C. E.—The author has given a very good presentation of the problem. The speaker agrees with Mr. Fowler as to the relative merits and proper use for bridges as compared with tunnels—that the tunnel is particularly suitable for public utility traffic, while the bridge is more appropriate under ordinary circumstances for private vehicular traffic.

There is one emphatic difference betweeen the New York-Brooklyn situation and that of Philadelphia-Camden which has not received the consideration deserved in the paper. At the time of the building of the Brooklyn Bridge—and of the later bridges between Manhattan and Brooklyn—Brooklyn was already a city of considerable size as compared with Manhattan, and entirely out of proportion to the present size of Camden relative to Philadelphia. Therefore, the increase of traffic resulting from the construction of the Delaware River Bridge should not be expected to parallel exactly that produced by the Brooklyn bridges. Brooklyn was—and still is—to a large extent a sleeping place for people whose business is in Manhattan: to some extent, Camden is similarly situated with relation to Philadelphia; but the fact that Brooklyn was already a large city when its bridges were built, gave it less opportunity for expansion, and, therefore, gave less possibility for transit development as a result of building these bridges than may be expected in the case of Camden and Philadelphia.

The benefit to be derived from a bridge is dependent on the service that it gives to the public; and, consequently, the development of traffic across the bridge is dependent on the improvement of that service. This is ample explanation for the scant increase in traffic on the Brooklyn Bridge on the intro-

* Cons. Engr., New York, N. Y.

† "Manhattan Elevated Railway Improvements", *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), p. 553.

‡ *Transactions,* Am. Soc. C. E., Vol. LXXV (1912), pp. 268–269.

duction of the shuttle service. If it is necessary to start at one edge of the river and stop at the other, and to take independent means of conveyance at either terminus, it breaks up the traffic too much for profitable use by the public.

With regard to the use to be made by public utility companies of the Camden-Philadelphia bridge, it seems self-evident that to obtain the greatest benefit for the public the Public Service Corporation of New Jersey, which feeds from and distributes to the New Jersey side, should be the company to bring passengers into and take them from the great city, rather than the Philadelphia transit lines that should carry their traffic just outside the city for distribution over the larger area. The latter would be of comparatively slight benefit to the public. The Philadelphia company probably would not receive enough benefit to warrant the extra expense, whereas the New Jersey companies would derive a material benefit, because they could carry their passengers to and take them from the heart of the city—a great boon to their patrons. It is fair to predict that such benefit would increase through a long period of years, and not simply be a temporary improvement followed by only slight progress. Such traffic controlled by the New Jersey Corporation, if properly managed and supervised, will grow increasingly as Camden and its vicinity develops.

This is another illustration of the old rule that as the population increases traffic will increase in proportion to the square of the population. The figures given by the author refer only to the population of Brooklyn; had they included the whole territory tributary to the Brooklyn Bridge instead of simply the Brooklyn end of it, probably the old rule would have found verification.

Ole Singstad,* M. Am. Soc. C. E.—The speaker would first like to refer to some of the points brought out in the discussion, before presenting certain ideas regarding the paper itself.

He heartily agrees with Mr. Fowler that, in the planning of the Delaware River Bridge, the vehicular traffic has undoubtedly been one of the determining factors, although this phase has not been discussed in the paper. He must disagree, however, on one or two other points. Mr. Fowler stated that tunnels are particularly well adapted for rapid transit purposes in a river crossing, but that there are physiological and psychological objections to tunnels for vehicular traffic. The speaker fully agrees with the first part of that statement, but not with the second part.

The exhaustive investigations carried out by the New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission, in connection with the construction of the Hudson River Vehicular Tunnel, have demonstrated that vehicular tunnels can be adequately ventilated at a reasonable cost, and that there need be no physiological objection to the use of tunnels for vehicular traffic any more than to the use of streets with heavy motor traffic.

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The psychological objections to vehicular tunnels are refuted by the experiences of the vehicular tunnels under the Thames in London, England, namely, the Blackwall Tunnel, which has been in successful operation for vehicular traffic for twenty-seven years, and the Rotherhithe Tunnel which has been in operation for sixteen years. These tunnels, 4 465 ft. and 4 930 ft. long between portals, respectively, are used by a mixed traffic consisting of horse-drawn, gasoline-propelled, and coal-burning steam-driven vehicles. Until about two years ago, the only ventilation was by natural draft through four shafts and the portals. Then a small mechanical ventilation plant was installed in the Blackwall Tunnel. In the summer of 1923, the speaker passed through these tunnels several times and found no evidence of any hesitancy to use them, even with the rather inadequate ventilation, and the engineer of the London County Council in charge of the operation of the tunnels stated that at no time had there been any interruption in traffic for this cause.

The speaker also disagrees with another statement made by Mr. Fowler, namely, that all tracks should be removed from the East River bridges and all available space on these bridges utilized for vehicular traffic, while the tracks should be carried in tunnels under the river. Such an arrangement appears advisable on its face, but would not be feasible in practice for the simple reason that, if the bridges were to be operated to their full capacity under those conditions, the existing approaches and adjacent city streets, or any approaches which could be created at a reasonable cost, would be overcrowded, and the resulting congestion would preclude the use of the bridges to their full capacity.

The paper is devoted principally to the discussion of the transit conditions at Philadelphia, Pa., and Camden, N. J., in relation to the Delaware River Bridge now under construction. The author has discussed several ways of utilizing the tracks of the bridge for transit purposes, but, in the speaker's opinion, has found no satisfactory solution of the problem. This is in no sense a criticism of the paper. The reason for this failure is more fundamental; such a solution was precluded when the type and location of crossing were determined. It appears that in deciding on these features, the requirements of vehicular traffic were given prior consideration to the transit requirements.

The delivery district for passengers in Philadelphia may be defined as a narrow rectangle about $1\frac{3}{4}$ miles long, extending from the Delaware River to 20th Street with Market Street as the east and west axis, and $\frac{3}{8}$ mile wide, extending from Arch Street, one block north of Market Street, to Walnut Street, two blocks south of Market Street. (See Fig. 7.)

It will be noted that the Philadelphia terminus of the bridge is entirely outside this district, being about $\frac{1}{4}$ mile north of its northern limit and near its eastern boundary. This means that under the best plan of transit operation suggested by the author, the "Camdenites" would not be delivered at or near their points of destination in Philadelphia. Further, the bridge terminus is not conveniently located on a transit line running longitudinally through the delivery district.

The author has stated that 77% of the entire passenger traffic between Philadelphia and Camden uses the Market Street Ferry. At the Camden terminus of this ferry are the Pennsylvania Railroad Station and the terminus of practically all the Camden surface lines. One-third of the passengers crossing on the Market Street Ferry from Philadelphia, that is, about one-fourth of the total traffic, are destined for the Pennsylvania Railroad Station, and a large part of those remaining take the trolley at the same place. The Camden terminus of the bridge is more than ½ mile from this point (see Fig. 7).

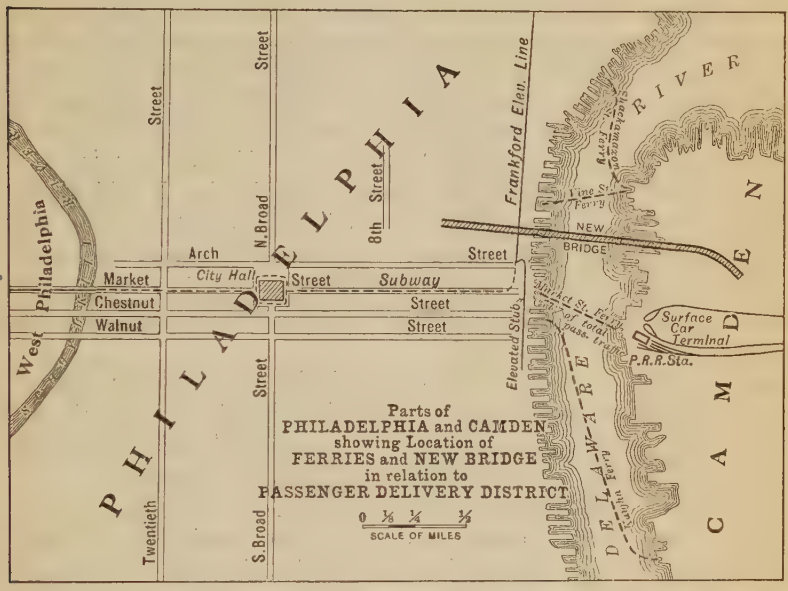

Fig. 7.

Evidently, as far as passenger traffic is concerned, the bridge is not located so as to meet the requirements on either side of the river, doubtless because in planning the crossing, the engineers had to consider the requirements of the vehicular traffic, the passenger traffic, the congestion on the city streets, and real estate values. All these conditions could not be met satisfactorily with a bridge.

If the bridge were located at a point where it would best serve the passenger traffic, that is, at or near Market Street, Philadelphia, and the Pennsylvania Railroad Station, in Camden, the resulting vehicular traffic congestion on the Philadelphia side would prove serious and the real estate damages would be high. In the speaker's opinion, the bridge is well located for vehicular traffic, but will offer little advantage over the present facilities for passenger transportation.

In 1918 and 1919, the speaker had occasion to study carefully the problem of a crossing between Philadelphia and Camden, both for passenger and

vehicular traffic. The traffic counts taken at that time showed that two rapid transit tracks would take care of the passenger traffic for a number of years, and that roadway facilities for two lines of vehicles in each direction would take care of the vehicular traffic for a corresponding period.

It appeared to the speaker then (and he is still of the opinion) that the most practicable crossing for passengers would be a rapid transit tunnel from a terminal station on the Camden side near the Pennsylvania Railroad Station and the terminus of the Camden trolley lines, under the Delaware River and connecting with a rapid transit line in Philadelphia running east and west through the delivery district. Under this proposal, the present Market Street Subway would split its service between Camden and Frankford, abandoning the elevated spur south along the Delaware River water-front; or probably better still, the tunnel would connect with a new subway through Chestnut Street, which, in turn, would connect near City Hall with the four-track subway to be built on North Broad Street. Thus, the majority of the passengers would be carried to their points of destination in Philadelphia or could proceed to any point served by the rapid transit system, and would also be carried to the logical point of transfer in Camden. If future conditions warranted, the rapid transit line could be further extended in Camden either as a surface or an elevated line. This would be the most satisfactory method of taking care of the passenger traffic.

The next step would be to provide facilities for vehicular traffic. The Philadelphia terminus for the vehicular crossing should be outside the delivery district for passengers, which is already badly congested. The site selected for the bridge is probably the one best suited for this class of traffic.

It is believed that the vehicular traffic alone could be cared for at considerably less expense by a tunnel than by a bridge, considering the long span of the bridge, 1 750 ft., and the low surface elevations on both sides of the river, which would favor a tunnel.

The construction at their logical locations of independent tunnels for both rapid transit and vehicular traffic would utilize the facilities to better advantage and with the expenditure of very little, if any, more money than a bridge for combined traffic. It is the speaker's opinion that this would be a much more satisfactory solution than the one adopted. It is true that the bridge provides four tracks; but this is more than is required, and the speaker agrees with the author that it is very questionable whether even two of the tracks will be operated to capacity.

The situation at Philadelphia-Camden, in the speaker's opinion, is an excellent illustration of the greater flexibility inherent in tunnel construction as compared with bridge construction for certain river crossings. The tunnel lends itself to providing the desired facilities without over-building and at the ideal location for each kind of service. On the other hand, in the case of a bridge, the tendency, for economic reasons, is to concentrate all traffic at one point, thereby causing congestion at the approaches and sacrificing the most desirable location for one class of traffic or the other.

JOHN A. MILLER, JR,* ASSOC. M. AM. SOC. C. E. (by letter).—By presenting data concerning the vehicular traffic on important river bridges, Messrs. Fowler, Steinman, Lewis, and Singstad have added valuable information to the paper and broadened its scope. The discussion of tunnels *versus* bridges has also shed light on the question from a new direction. Both these important considerations were omitted from the original investigation because the study was made after the erection of a bridge in a particular location had been definitely decided upon, and because the problem was viewed only from the railway standpoint.

As Mr. Singstad has pointed out, the location of the Delaware River Bridge is probably not ideal as far as passenger traffic by rail is concerned; on the other hand, to build a bridge where it would best serve the rail traffic would be impracticable on account of the resulting vehicular congestion at both ends. There is, therefore, much to recommend a tunnel. Even in that case, however, a problem would be presented concerning the best way to operate the facilities provided, and the conclusions already reached would be applicable.

The objections raised by Messrs. Weir and Steinman to placing railway tracks on the bridge appear to exaggerate the importance of the automobile in the general transportation scheme. In the discussion it was said that "the capacity of the street is vastly increased by eliminating the trolley car." It was suggested that the capacity of a highway bridge would be increased if railway tracks were omitted from the design. The fact is, however, that the electric railway car, as far as efficient use of the street is concerned, surpasses any other means of transportation. Traffic counts recently made in six important cities show that the street cars are carrying upward of 75% of the total traffic, although they constitute only about 15% of the total number of vehicles. The figures are given in Table 1, from which it will be seen that the elimination of the electric railway car would increase the capacity of the street for automobiles, but would seriously reduce it for general transportation purposes. There seems to be no good reason why the automobile should be especially favored in this respect, either in the street or on a bridge. The worth of a transportation facility on a bridge should be judged by the service it performs rather than by the tolls it provides.

The statement that the original cost of a bridge such as that being built across the Delaware River, could be reduced as much as 50% by the elimination of railway tracks is true only in case the railway passengers elect to travel by an entirely different route. They could not be carried over the bridge by any other means that would produce as small a live load. The total weight per passenger carried, which is probably the best basis on which to compare the live load capacity required for various types of transportation units, is so nearly the same for a modern street car and an ordinary single-deck bus, that there is little to choose between them. Among the passenger automobiles, only a fully loaded Ford can compare with either of them in economy of weight per passenger. Typical weights are given in Table 2.

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TABLE 1.—Traffic Statistics.

	Composition of Traffic.		Passengers Carried.	
	Railway cars. Percentage.	Other vehicles. Percentage.	By railway. Percentage.	By other vehicles. Percentage.
Chicago, Ill.	10	90	74	26
Baltimore, Md.	12	88	89	11
St. Louis, Mo.	10	90	75	25
Los Angeles, Calif.	25	75	92	8
Atlanta, Ga.	9	91	72	28
Montreal, Que., Canada	25	75	93	7

Probably none of these facilities would be operated at its most efficient loading, except during the rush hours. Their relative merits, however, would not be altered on that account.

TABLE 2.—Weights of Transportation Facilities.

Class of facility.	Total weight, in pounds.	Passenger capacity.	Pounds per passenger.
Street Railway Cars:			
Birmingham Electric Company	33 800	110*	308
Connecticut Company	30 000	100*	300
Durham Public Service	27 000	90*	300
Eastern Texas Electric Company	26 000	85*	306
Northwestern Ohio Railway and Power Company	34 500	95*	363
Philadelphia Rapid Transit Company	34 000	100*	340
Motor Buses:			
Fageol	9 600	40†	240
Fifth Avenue, J.	8 235	40†	206
Garford, 51-D.	10 000	40†	250
Mack (low bus)	9 200	32†	288
White, 50-A	8 400	32†	262
Yellow, Z-29	10 000	40†	250
Passenger Automobiles:			
Ford Touring	1 650	5	330
Dodge "	2 546	5	509
Buick "	3 120	5	624
Nash "	3 400	5	608
Cadillac "	4 025	7	575
Packard "	4 746	7	678

* Estimate based on railway company's specifications.
† Estimate based on manufacturer's specifications.

The slight advantage in weight possessed by the bus is more than counterbalanced by the road space occupied. This is a matter of paramount importance in the case of a bridge where the roadway is likely to be congested. The ordinary electric railway car occupies approximately 3.5 sq. ft. per passenger carried, a single-deck bus of average size about 5.0 sq. ft. per passenger, and a private automobile somewhat more than 20 sq. ft. per passenger. If every inch of space occupied by electric railway tracks on the East River bridges in New York, were devoted to passenger transportation by automotive vehicles, the passenger capacity would be woefully insufficient.

Moreover, experience has demonstrated times without number that traffic movement is accomplished most expeditiously with large units. In other words, 50 railway cars will cause far less confusion and congestion than 100 or more busses, regardless of the street space occupied, relative weights, or other details. Under these circumstances, the conclusion appears inevitable that if a bridge is to be used for mass transportation of passengers the means should be an electric railway.

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Paper No. 1555

THE ECONOMICS OF HYDRO-ELECTRIC DEVELOPMENT*

By Daniel W. Mead,† M. Am. Soc. C. E.

With Discussion by Messrs. L. F. Harza, Harry A. Hageman, C. P. Dunn, Charles B. Hawley, W. S. Lee, Brent S. Drane, and Daniel W. Mead.

Synopsis

The main question involved in the consideration of any proposed hydro-electric development is: Can energy be supplied from the proposed development to a market at such times, in such quantities, and at such prices, that it will command the market and assure a suitable return on the total cost of the development? The answer to this question depends on a multitude of factors any one of which may be so unfavorable as to require a negative answer or, if overlooked, to result in the failure of the project.

These factors must be sub-divided, in order fully to cover the numerous questions which must be considered for a full analysis of any project. This has been attempted in a fairly complete manner in Appendices A, B, and C.

Of the points discussed, the most important are, perhaps:

1.—The difficulties and expense involved to an independent development, in entering a market already served, in part at least, by power from some other source, and the resulting low price at which the hydro-electric power must often be sold.

2.—The great variations which commonly occur in the flow of streams, the difficulties in determining the average and extremes of these variations, the resulting irregularity in the power supply from such streams, the low value of the resulting irregular power supply, the necessity for auxiliary power to render the total power developed firm power, the economical point of develop-

* Presented at the Spring Meeting, Atlanta, Ga., April 9, 1924.

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ment of such combined power, and the advantages of hydro-electric developments by power companies having a business already established.

3.—The handicap to the economical development of an independent hydro-electric project, due to the heavy cost of financing a somewhat speculative enterprise.

4.—The sources of expense to an independent development in establishing a business, due to early lack of income which necessitates deferred dividends and the payment of fixed charges and, perhaps, of operating expenses from other than income sources; the necessity of provisions to meet development expenses; and a method of estimating the financial operation and the expense of market development.

5.—The necessity for accurate estimates of income and expense. The recognition of hazards which may develop, and the necessity of provision for such hazards by a somewhat excessive possible balance in favor of the project, or its abandonment.

The first question of economics involved in the consideration of any hydro-electric development is that of expediency. Is it desirable that the development be undertaken?

The success of any hydro-electric development depends on the possibility of supplying energy in such manner and at such a price, that it will command a market and assure a fair return on the cost of the development. This principle is easily expressed and easily understood, but is difficult to apply to any concrete case. A favorable answer to this question depends on many factors which may be classified in a great variety of ways, but which are herein considered under the following headings:

1.—On the acquisition of suitable legal rights.
2.—On the command of a satisfactory power market.
3.—On reasonably suitable physical conditions.
4.—On favorable hydrological conditions.
5.—On the proper design of the works involved.
6.—On economical and substantial construction.
7.—On economical financing.
8.—On rapid and economical development of business.
9.—On economical management, operation, and maintenance.
10.—On accurate estimates of power output and income and of cost of promotion, financing, construction, depreciation, development, management, operation, and maintenance.

These factors will vary in importance with each possible development. Any one of them, however, may be so unfavorable as to be fatal to the economic success of any particular project. There will never be a development in which all these factors are so favorable as to leave nothing to be desired; hence, economic success will be assured only when the quantity of power available and the market are such as to provide for the expense of overcoming any and all unsatisfactory conditions.

The attempt will be made in this paper to discuss each factor briefly and with particular reference to those elements that have proved to be of most vital importance or have been most commonly overlooked or under-estimated.

1.—The Acquisition of Suitable Legal Rights

The necessity of franchise or legal rights under the Federal or State laws to build and to operate a hydro-electric development is obvious and needs no further mention. Attention must be given to the State water laws which differ widely, from the English principle of riparian ownership in the East, to the entire separation of land ownership and water rights in the West.

The acquisition of flowage rights and rights of way is always of importance. With the low-head plants of the East, constructed on rivers which flow through populous farming districts and with which considerable pondage or storage is necessary to equalize flow or to take care of the load factors, the acquisition of legal rights and rights of way may be a troublesome and uncertain process, the expense of which is difficult to determine accurately in advance. With high-head plants, however, in mountainous country, unfit for settlement and agriculture, the storage required may be comparatively small and the legal rights simple to determine and obtain.

For a private development, no condemnation proceedings are possible in most States, and site, flowage, and rights of way may be purchased only if owners are willing to sell. In some States, even public utilities are not given condemnation rights or are given only limited rights; and when the necessary properties are in unfriendly hands, exorbitant charges for property sometimes lead to an excessive cost for the project as a whole. Lands condemned often are given high valuations, as the sympathy of Courts, and especially of jurors, is usually with the private citizen and adverse to corporations.

A further point of importance, as a probable item of expense in many developments in settled parts of the country, is the excess flowage land that may have to be acquired, or the damages, real or imaginary, that may have to be paid in connection with flowage. One of the most difficult problems to solve with exactness is the height of back-water, caused by a dam under the different conditions of stream flow, and the resulting limits of flowage. Even when the flowage can be determined with fair accuracy, it is often impossible to establish the facts to the satisfaction of Court and jury. When an unusual flood occurs soon after the construction of a development, claims for damages to land, which will involve either purchase or the payment of damage, are likely to be raised. One successfully prosecuted claim of this kind may give rise to many other claims. Frequently, the expense of purchase, damages, and litigation concerning such lands becomes a considerable item.

2.—The Command of a Satisfactory Power Market

The difficulties of determining market conditions and the price for power which a development can command in such market, vary widely for different developments. If the development is a part of an operating system, the market will already have been established, the price obtainable for power will be certain, and all market conditions known. However, when a new development

has to enter a market already supplied, in part at least, by other forms of power, and especially where power is furnished from a central station operating profitably under the conditions that have previously existed, it often becomes exceedingly difficult to induce the operating company to buy and distribute power from the hydro-electric plant at a price which will be equitable and profitable to the new development. Almost uniformly the price that can be obtained under these conditions will be much below any estimated price for power that the hydro-electric interests are likely to anticipate.

When a market must be developed for a new hydro-electric plant, extended examinations and investigation are necessary. Is the demand for power sufficient to warrant the development? Can a price for power be obtained that will warrant the necessary investment?

A census of power users will determine the amount of power used in the territory considered. The determination of the price at which power can be sold is a more difficult problem. The actual value of power in any community where power is a necessity, is the cost of developing such power by the means ordinarily used, or which ordinarily would be used under the conditions and in the quantities used, or to be used, in the particular location. This is the upper limit at which power from a new hydro-electric development can be sold, and such price can be obtained only when the market is practically undeveloped by other sources. The lower limit is the actual cost of the power to the hydro-electric company. Between these limits, there is frequently a wide range in values. The higher price must not be so great as to discourage the use of power or as to induce large users to undertake its generation rather than to contract for its purchase.

When the market is already supplied with power generated by fuel-burning plants, the sale price for hydro-electric power must be lower than the cost of fuel-generated power, otherwise, users will continue to generate their own energy. With the developed market, the higher price previously referred to is so high that, for commercial reasons, such a rate cannot obtain. At this price, the customers would pay a rate equal to the actual cost of power generated by their own plant, and there would be no inducement for them to become customers of the hydro-electric development. The higher price is, therefore, a commercial impossibility. The lower rate, the actual cost of hydro-electric power, is equally impossible, for such rates would not encourage hydro-electric development. The public, including all consumers of power generated by a hydro-electric plant, will from commercial necessity receive a portion of the benefit resulting from the use of such power, from the commercial conditions that follow its development. Rarely, if ever, can a hydro-electric development obtain the higher return for its output. Such a development, as a rule, must supply power to a market which is, partly at least, supplied with power from some other source. Investments in power-generating machinery of some kind have been made. Fixed charges have been incurred, and, in order to introduce its product, the hydro-electric development must sell power below the station cost of fuel-generated power, and not on the basis of fixed charges plus operating expenses of such plants, which has been the true measure of the actual value of power to the date of its introduction from the new source. Only

where the market developed is entirely new, and where no fixed charges have been entailed for previous power-plant installation, can a hydro-electric development realize from the sale of power even a part of the fixed charges of the steam plant. Even under such conditions, a material reduction must be made, in order to induce customers not to install isolated plants of their own for the production of such power, but to purchase the power developed from the hydro-electric plant.

For example, consider a community of 30 000 or 40 000 in the interior of the country. A well planned steam-electric plant working under the conditions which ordinarily obtain in such communities, can develop power at its switchboard for about 1.6 cents per kw-hr. Of this amount, approximately 0.5 cent per kw-hr. is fixed charge and 1.1 cents per kw-hr. is operating cost.

Assume that hydro-electric power can be delivered at the city stations at a cost of 0.6 cent per kw-hr. If a hydro-electric company sells its output to the steam-electric company, it will be found that the fixed charges, maintenance, and some of the operating expenses on the steam plant will continue, as the steam plant must be maintained ready for almost instant use, in case of failure of the transmission line, in order to assure the service to which the community is entitled. There is no immediate economy to the steam-electric company in the purchase of water-generated power unless it can be purchased at less than the operating cost of steam power. Consequently, to effect a sale, the water power must be sold at 0.7 to 1 cent per kw-hr., delivered at the customer's switchboard. The higher price would probably not be regarded as attractive to the steam-electric company, and the lower price would seldom be attractive to the hydro-electric company and might involve a direct loss, unless the total current sold to all customers constitutes a large proportion of the capacity of the plant. If a direct combination can be effected between the water-power company and the steam-electric company, the steam plant may be utilized as an auxiliary to the water power, and the whole value of the output utilized by the combined interests. This combination is usually the only way in which a larger net profit can be obtained from a water-power development. The best results can be obtained only by combination with an industry or market already developed in which the power can be utilized at its true market value.

In considering the influence of the market, it should also be noted that the load factor, on the basis of which power must be sold, is an important element in the cost of hydro-electric development. A combined load approximating a load factor of 100%, which is seldom possible, involves a minimum of expense in machine installation. A load factor of 50% practically doubles the cost of machinery and greatly increases the cost of the power-house, and a load factor of 33⅓ or 25% triples or quadruples the cost of the machine installation.

3.—Reasonably Suitable Physical (Topographical and Geological) Conditions

Physical conditions at the dam and power-house sites are sometimes easily determined, but adequate examination frequently entails considerable delay and expense. At the preliminary inception of a project, it may be exceedingly important to limit to a minimum the expenditures for examination, and proper

physical examination may be delayed until other conditions seem to demand immediate and favorable action, and the project is launched without a complete and satisfactory physical examination having been made. If unfavorable conditions, not readily foreseen, develop during construction, the results may be disastrous, if not financially fatal.

The physical conditions at the site of the dam, the station, and appurtenances are among the most important factors affecting the cost of construction. A poor foundation may prove fatal to the success of the project, as in the case of the municipal plant at Austin, Tex., or may involve a large and burdensome expense as, for example, the Hales Bar development on the Tennessee River. Thorough and detailed examination prior to construction would probably have led to a knowledge of these conditions, and to a corresponding change in location or in plans, or the abandonment of the project. The works must be designed to fit and to develop the physical conditions to the best advantage, and the physical conditions must be studied with regard to the particular works designed to develop them.

4.—Favorable Hydrological Conditions

Where hydro-electric plants are entitled to take a limited water supply from great rivers, the conditions of stream flow may be definite and well established. In the development of any ordinary stream to the point of maximum economy, the great variations of flow which normally occur become important matters. These variations in stream flow, the consequent variations in head, and the resulting possible power output, must be considered with reference to market conditions.

Although there are some notable exceptions, the market in general requires the continuous delivery of power. The annual load will vary somewhat due to the season. Some loads are less in winter and greater in summer; others are greater in winter and less in summer; and some are subject to hourly, daily, or seasonal variations. These loads vary somewhat from year to year, usually increasing in growing communities, and fluctuating according to commercial activities; but the general form of load curve remains quite the same in character for a given community.

The water power which can be developed from a given stream is also subject to considerable variation, due to the character of the stream and the climatic conditions peculiar to each particular drainage area. Usually, there are one or two seasons of high water and of low water during each year. The high water in most streams will occur generally in the spring, and sometimes, also, in the fall. The dry season of summer, together with the demands of vegetation, produces low water, and in northern streams, the freezing of the sources commonly results in even lower water during January and February. In addition to these variations, the years differ widely. Occasionally, there are years of fairly continuous high flow whereas other years have equally continuous low water, and these periods rarely bear any relation to the demands for power.

No one unfamiliar with hydrological studies will realize the great variations in the flow of streams from year to year. Fig. 1 shows the daily stream

flow of a single river (not an extreme case) for the maximum and minimum years during the last eleven years.

All hydrological studies with reference to power output can be made to greater advantage by platting the data in the form of duration curves, which will show the duration of each intensity of flow for each year without reference to date of occurrence (Fig. 2). From all the data available, for the period of records, an average duration curve can be platted (Fig. 2), from which the average quantities of power that could have been developed from the stream during the period of record can be determined. From a study of Fig. 2, using the flow in cubic feet per second per square mile of drainage area as the basis for power study, it will be noted that during the year of minimum low-water flow (1919), power would have been available on the stream in question for 98.5% of the time, to a flow corresponding to 0.25 sec-ft. per sq. mile; during the lowest year (1923) to 0.3 sec-ft.; during the average year to 0.35 sec-ft.; and during the highest year to 0.51 sec-ft. In general, therefore, the hydro-electric development is confronted with the conditions of a superabundance of power at certain seasons and for certain years, and with a deficiency of power at other seasons and for other years. These conditions must be met in one of the following ways:

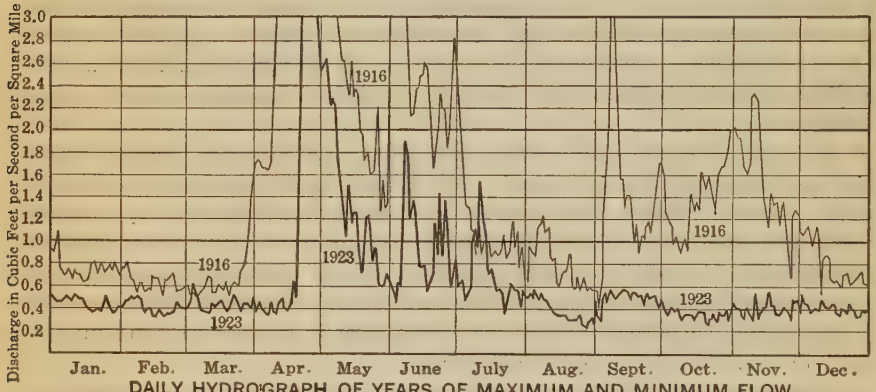

FIG. 1.

First.—By the development of the stream only to a minimum flow. In general, this will entail so great an expense for such a small amount of power output as to render any project based on such a flow entirely uneconomical.

Second.—By the sale of firm or primary power to certain customers, and of surplus or secondary power to other customers who can use power when it is available, or who can utilize their own power plants or other sources of power during periods when power from the hydro-electric plant is deficient.

Third.—By the establishment of fuel-burning auxiliary stations from which power will be available whenever a water shortage occurs. By this method, the power from a hydro-electric development can be made firm power under all conditions of river flow for which the development is designed.

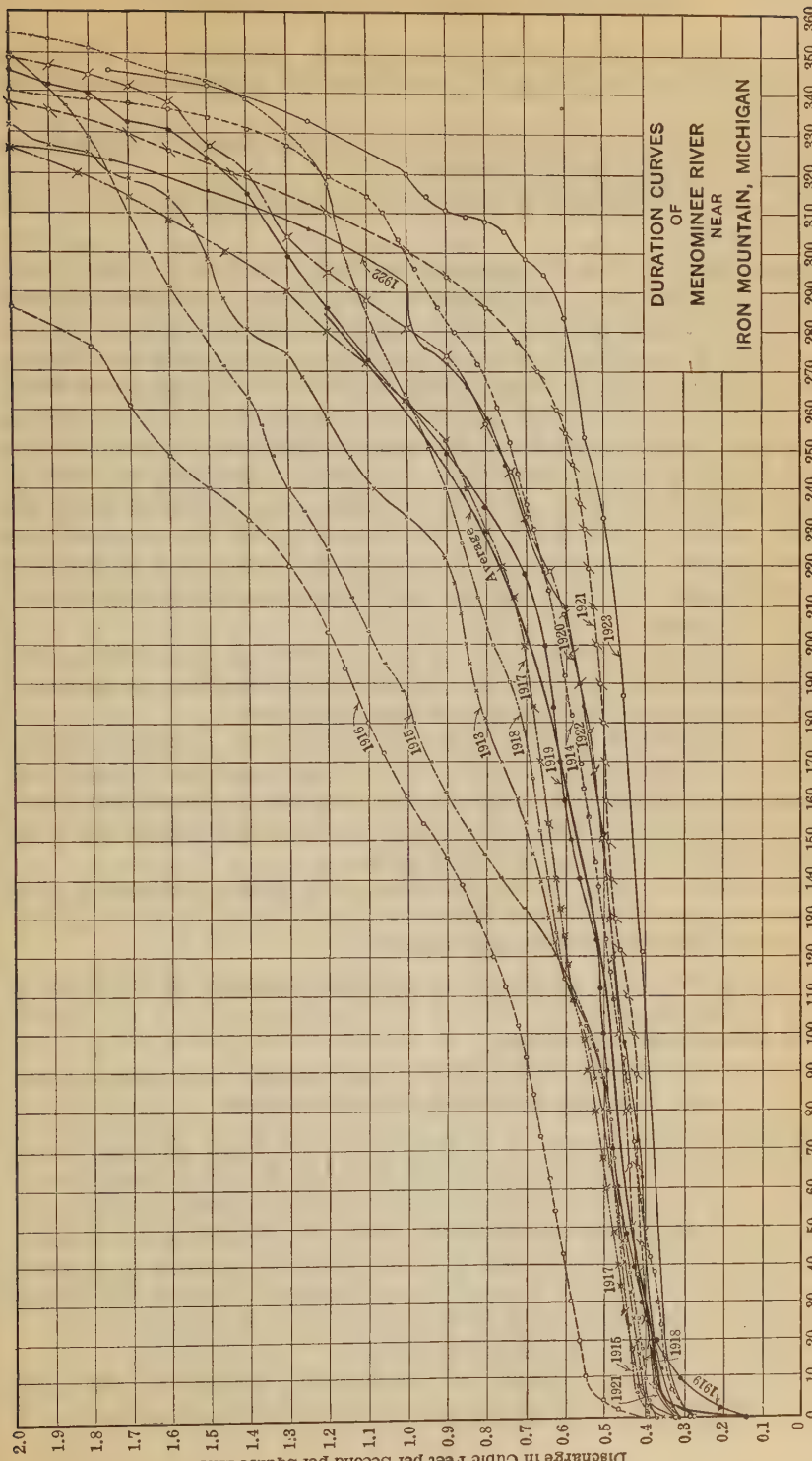
DURATION CURVES
OF
MENOMINEE RIVER
NEAR
IRON MOUNTAIN, MICHIGAN
Discharge in Cubic Feet per Second per Square Mile
Number of Days
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
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FIG. 2.

Fourth.—By the construction of reservoirs to store the flood waters for use during periods of low flow, thus often substantially increasing the minimum power output.

Fifth.—By interconnecting hydro-electric plants on streams with low water occurring at different seasons of the year.

Sixth.—By a combination of the methods outlined under the third, fourth, and fifth items.

Surplus power is often of comparatively small value and, if it can be sold at all, can command only a low price, whereas firm power is uniformly of much greater value. For example, for many years the grinding of pulpwood has been considered one of the ways of using surplus power to advantage. During the season of high water, an excess of pulp is manufactured, lapped, and stored for use during low water. This process requires the product to be pressed, dried, stored, and beaten, when used for paper-making, involving an expense for lapping, handling, storing, deterioration in storage, re-handling, and beating, which amounts to $5 to $7 per ton. Therefore, even if surplus power for grinding can be obtained as low as 0.35 cent per kw-hr., the paper-mill can often better afford to pay from 0.75 to 0.95 cent per kw-hr. for firm power for grinding the pulp as it is needed, pumping it to the paper machines without additional processes.

The method of developing water power in excess of the minimum flow, although uneconomical, as entailing a low price for much of the power output, has yet been found practical for some independent plants.

The construction of auxiliary plants involves a large additional expense, but materially increases the value of the hydro-electric output. Which of the methods of development just outlined will be best for an independent hydro-electric development is a question of character of market and price for power that must be determined for each particular case.

The study of the hydrology of the stream is of vital importance in every case, and in many it is the most difficult to analyze of all the economic factors involved. These difficulties arise from:

1.—The lack of data on the stream, and especially at the point of development.
2.—The inaccuracy of data.
3.—Lack of knowledge as to whether data available even when accurate, fairly represent the average future stream flow.

Although a large amount of stream-flow data is being obtained by the U. S. Geological Survey, it often happens that data are not available for the particular stream studied and they are still less frequently available for the particular site of a proposed development. Even if information for a period of years is available, the accuracy of stream-flow data remains to be determined; and if the accuracy is unquestioned, there still remains the problem of how closely the record for a given term of years represents the flow to be expected during the life of the development.

When flow records for the site are not available, the data for other sites must be used as a basis for an estimate at the site under consideration. If

no data are available for the stream, data from other neighboring streams must be used. The transfer of data or the determination of the flow of one stream from that of another is of doubtful accuracy, even if the streams are closely adjoining. This great variation in adjacent streams is illustrated by Fig. 3, which shows the comparative daily hydrographs of the Hudson, Oswego, and the Genesee Rivers in the State of New York.

Table 1 shows the possible inaccuracies in the data, derived from even a considerable period of records, when averaged for forecasting the future. This table shows the maximum and minimum variations in average annual stream flow from the means of certain fairly extended records, and the more pertinent data, for the study of water power, of the variations from average flow for the minimum six months of the flow of various streams. It will be noted from Table 1 that the lower part of the average duration curve shown in Fig. 2 covering an 11-year period, might vary approximately $\pm$ 18% on the basis of the record of the Merrimack River, $\pm$ 35% on the basis of the Croton River, $\pm$ 13% on the basis of the Tennessee River, $\pm$ 8% on the basis of the Columbia River, and $\pm$ 40% on the basis of the Colorado River of Texas, according to the location and character of the stream.

Comparative studies of rainfall for the years during and previous to the years of stream-flow record will often indicate with considerable accuracy the possibility of great variation from the recorded average flow of any stream.

Owing to lack of space, discussion of hydrology must be confined to the foregoing statements, which are designed merely to call attention to the economic importance of the subject and to the difficulties frequently involved in reaching correct conclusions from the data usually available.

After a duration curve is established, that is believed fairly to represent the future average flow of the stream, the economical point of development can be determined approximately by a method devised by F. E. Turneaure, M. Am. Soc. C. E. This analysis is shown in Appendix D.

5.—The Proper Design of the Works Involved

As this factor cannot adequately be discussed in a paper of reasonable length, only a few important points will be mentioned; but other points are noted in the more complete outline of Appendix A. The factors outlined in the beginning of this paper of necessity overlap to a certain extent; under the heading, "Reasonably Suitable Physical Conditions", the question of foundations has already been briefly considered. This question is one of judicious selection as well as physical condition, as conditions sometimes vary greatly within comparatively narrow geographical limits.

It is of great economic importance that structures shall be designed to provide for the maximum floods likely to occur, and this again reverts to the study of the hydrology of the stream. Occasional floods may occur that greatly exceed those recorded, even in long-time records of maximum stream flow. To illustrate this point, reference may be made to Fig. 4, which shows a view of the reservoir and dam of the Oklahoma, Okla., Water-Works. A flood of about 133 000 cu. ft. per sec. washed around the end of this dam during October, 1923. The dam and flood-control works, which have a drain-

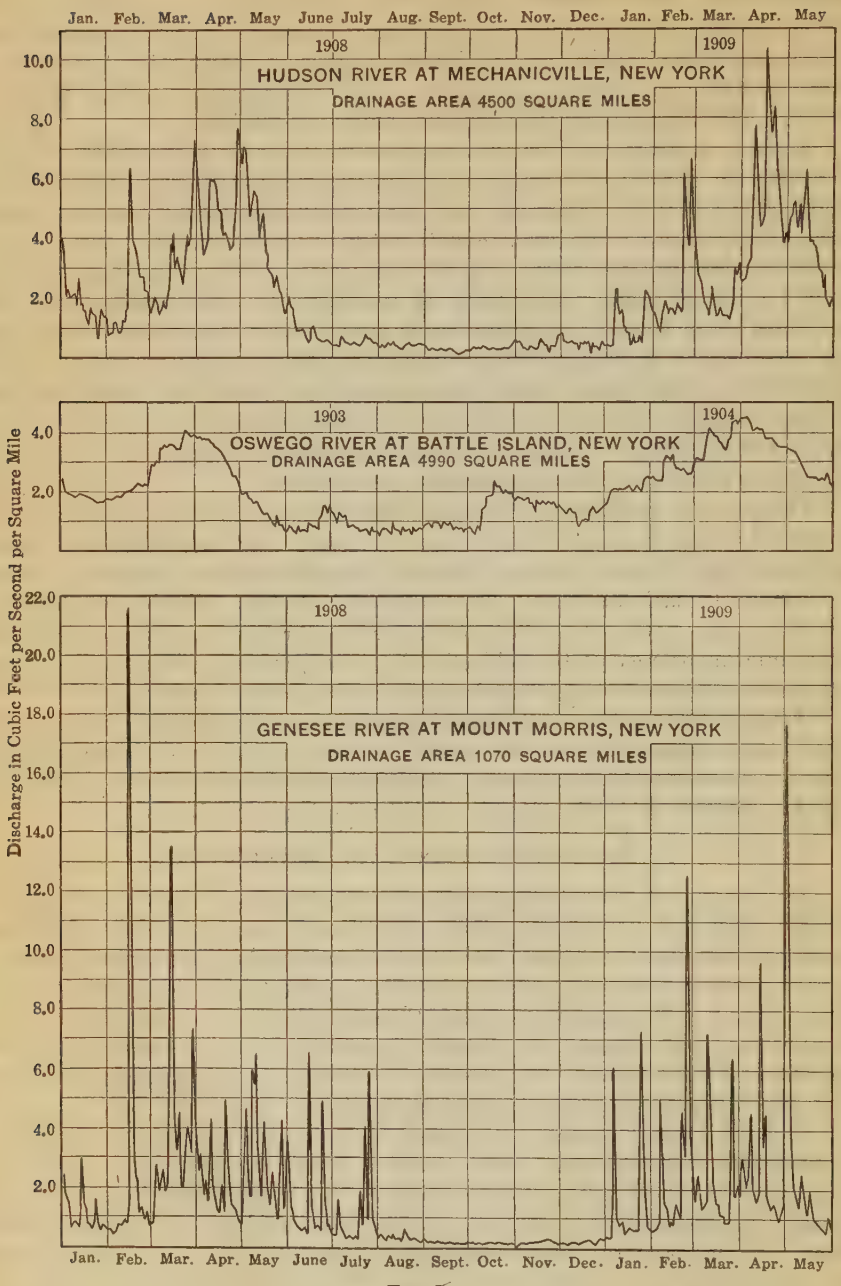
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May
1908
1909
HUDSON RIVER AT MECHANICVILLE, NEW YORK
DRAINAGE AREA 4500 SQUARE MILES
10.0
8.0
6.0
4.0
2.0
1903
1904
OSWEGO RIVER AT BATTLE ISLAND, NEW YORK
DRAINAGE AREA 4990 SQUARE MILES
4.0
2.0
Discharge in Cubic Feet per Second per Square Mile
1908
1909
GENESEE RIVER AT MOUNT MORRIS, NEW YORK
DRAINAGE AREA 1070 SQUARE MILES
22.0
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May

FIG. 3.

TABLE 1.—MAXIMUM AND MINIMUM VARIATION IN AVERAGE STREAM FLOW FROM MEAN RECORD.

Rivers.	Years of record.	Number of years.	Depth, in inches, mean flow.	1-Year Departure.		5-Years Departure.		10-Years Departure.		15-Years Departure.		20-Years Departure.		25-Years Departure.		30-Years Departure.		35-Years Departure.		40-Years Departure.		45-Years Departure.		50-Years Departure.		55-Years Departure.		60-Years Departure.	
				+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−
Lake Champlain Outlet, Richelieu River	76 16	41	20.46	29.10	29.48	17.88	15.46	8.34	9.00	7.40	1.14	6.25	1.48	5.35	0.84	3.65	+0.98	2.04	+0.29	0.62	0.51	....	....	....	....	....	....	....	....
Penobscot River, at Outlet at Sebago Lake, Maine	87 19	33	20.55	46.92	52.74	27.62	25.34	11.35	20.13	11.45	14.42	6.60	9.08	0.70	7.32	0.70	2.40	.....	.....	....	....	....	....	....	....	....	....	....	....
Sudbury River, at Framingham, Mass	75 16	42	20.92	70.89	48.33	37.32	36.04	18.26	24.52	16.78	15.44	14.96	6.98	9.22	6.64	8.60	+0.68	4.40	0.76	0.39	0.34	....	....	....	....	....	....	....	....
Lake Cochituate Basin, near Cochituate, Mass	64 18	55	19.16	89.25	53.03	46.00	31.05	18.15	20.04	13.50	16.23	11.80	8.50	9.30	7.93	6.60	2.08	6.05	4.12	6.05	3.80	3.95	2.92	0.83	1.20	....	....	....	....
Merrimack River, at Lowell, Mass	49 60 67 16	62	26.74	29.24	29.40	21.00	19.41	16.45	13.80	11.51	10.44	8.45	8.08	6.35	7.22	5.80	6.14	2.42	5.35	0.90	4.52	0.71	2.28	0.48	0.82	0.00	0.86	0.00	0.22
Merrimack River, at Lawrence, Mass	50 08	59	22.62	30.78	37.27	21.82	18.34	10.32	15.34	5.70	9.31	5.62	6.76	5.12	5.08	3.52	3.93	1.71	4.59	2.30	2.34	1.07	2.34	0.75	1.85	1.10	1.18	....	....
Merrimack River, at Lawrence, Mass	80 18	39	20.02	44.70	46.79	33.50	24.10	15.05	19.80	11.56	12.16	11.40	5.02	7.97	4.17	3.67	0.28	1.32	0.48	....	....	....	....	....	....	....	....	....	....
Croton River, at Old Croton Dam, N. Y	68 21	54	22.57	51.70	48.20	27.95	26.36	16.25	17.68	9.91	14.14	10.00	9.97	8.55	7.27	6.60	5.94	5.98	1.51	3.05	1.68	0.83	1.99	0.00	1.77	....	....	....	....
Tohickon Creek, at Point Pleasant, Pa	84 13	30	27.48	54.10	45.15	21.80	16.84	13.20	12.65	4.90	10.36	2.50	8.20	1.00	3.22	.....	.....	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....
Lake Ontario	60 07	48	12.01	19.25	24.73	14.65	15.48	9.75	12.08	7.25	9.82	5.60	7.34	4.82	4.24	4.00	3.58	3.75	3.16	1.40	2.33	0.32	1.08	....	....	....	....	....	....
Lake Superior	60 07	48	14.62	23.65	20.66	14.10	14.84	10.05	9.30	8.28	8.00	7.50	6.84	5.68	6.30	4.15	5.68	1.44	5.68	0.95	1.78	0.13	1.16	....	....	....	....	....	....
Wisconsin River, at Merrill, Wis	04 20	17	16.78	50.05	42.97	19.60	13.52	8.15	7.15	1.90	6.44	.....	.....	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....	....	....	....	....
Tennessee River, at Chattanooga, Tenn	75 17	43	24.82	49.75	49.48	19.45	18.49	14.21	7.57	8.73	6.97	5.96	6.61	3.70	6.24	2.65	3.64	2.38	1.53	0.00	1.73	....	....	....	....	....	....	....	....
Tennessee River, at Florence, Ala	95 17	23	22.69	31.50	44.64	14.00	11.18	3.10	2.67	3.29	1.66	1.00	2.05	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....	....	....	....	....
Coosa River, at Riverside, Ala.	97 16	20	22.65	38.65	57.93	18.00	11.78	8.36	2.50	4.51	1.53	.....	.....	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....	....	....	....	....
Columbia River, at The Dalles, Ore	79 16	38	13.03	47.83	42.77	27.98	20.06	16.08	13.91	7.02	11.70	7.49	8.24	6.49	4.63	3.42	4.71	1.35	3.10	....	....	....	....	....	....	....	....	....	....
San Gabriel River, at Yuma, Ariz	96 18	23	10.60	178.70	92.17	62.45	74.05	47.45	53.96	33.20	17.36	9.80	5.20	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....	....	....	....	....
Red River of the North	82 19	38	1.09	109.90	74.22	35 35	50.28	22.45	25.42	17.85	9.76	11.40	4.23	9.60	3.30	5.00	7.00	3.12	7.92	....	....	....	....	....	....	....	....	....	....
Colorado River, at Yuma, Ariz	79 17	39	14541*	146.20	81.23	98.00	64 38	76.90	56.66	69.00	51.35	45.14	47.14	28.90	41.20	16.46	23.04	7.78	10.12	....	....	....	....	....	....	....	....	....	....
Colorado River, at Austin, Tex	99 22	24	0.96	187.50	77.06	56.25	52.06	27.10	31.24	6.25	21.87	0.00	17.70	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

*Cubic feet per second.

TABLE 1.—(*Continued*).

VARIATIONS OF FLOW FOR THE MINIMUM SIX MONTHS.

Rivers.	Years of record.	Number of years.	Depth, in inches, mean flow.	1-YEAR DEPARTURE.		5-YEARS DEPARTURE		10-YEARS DEPARTURE.		15-YEARS DEPARTURE.		20-YEARS DEPARTURE.		25-YEARS DEPARTURE.		30-YEARS DEPARTURE.		35-YEARS DEPARTURE.		40-YEARS DEPARTURE.		45-YEARS DEPARTURE.		50-YEARS DEPARTURE.		55-YEARS DEPARTURE.		60-YEARS DEPARTURE.	
				+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	−	+	-
Merrimack River, Lawrence, Mass.	80	19	39	4.88	72.55	50.41	50.60	32.78	21.70	18.03	15.78	12.70	12.70	8.00	9.00	6.56	4.30	0.40	1.64	0.40	....	....	....	....	....	....	....	....	....	....
Croton River, at Old Croton Dam, N. Y.	68	21	54	8.94	110 16	76.25	49.08	42.03	36.75	33.42	23.55	27.70	27.22	16.29	16.71	9.91	13.48	5.10	10.56	+0.14	5.51	1.34	1.17	2.52	0.05	2.64	....	....	....	....
Tennessee River, at Chattanooga, Tenn.	75	17	43	6.43	79.69	52.91	30.71	24.46	14.55	12.64	6.31	8.24	4.12	4.56	6.78	4.10	3.82	3.32	2.72	0.22	0.00	1.46	....	....	....	....	....	....	....	....
Columbia River, at The Dalles, Ore.	79	16	38	2.91	43.40	27.78	14.17	14.02	9.35	8.53	4.19	7.15	4 19	2.34	4.19	2.34	1.79	3.02	0.41	3.10	....	....	....	....	....	....	....	....	....	....
Colorado River, at Austin, Tex.	99	22	24	0.33	438.25	78.46	84.60	60.00	38.45	47.69	10.77	35.38	4.60	26.15	.....	.....	.....	.....	.....	.....	....	....	....	....	....	....	....	....	....	....

age area of 12 000 sq. miles tributary to them, were designed to pass about 18 000 cu. ft. per sec. This was much in excess of any recorded flow, the records being very limited. A study of the hydrology of the stream leads to the conclusion that no flood equal to that of October, 1923, has occurred in fifty years, but that even a greater flood must be expected.

In the provision for floods, a point of much economic importance should be noted. If excessive floods are provided for by an overflow spillway, flowage lands to a considerable height above the crest of the dam must be purchased to be utilized only during floods. With many plants, these lands may be utilized to increase the operating head and pondage by the use of gates in the dam instead of an open spillway. These gates will keep the head-waters at a maximum elevation during low water and provide high discharge capacity at times of flood.

The provision of pondage so that during days of average flow, water may be stored during periods of low power consumption and used during the peak demand without too great a reduction in head, is another important economic problem, especially with a low load factor. In general, it may be stated that plants should be designed with the human equation always in mind. As far as possible, the arrangement should be such that the right thing is the easiest thing to do. Considerable expense is warranted if operating costs can be reduced. With this principle in mind, automatic, semi-automatic, and remote controlled stations must be mentioned.

Efficiency in operation is also of much importance. High efficiency is frequently sacrificed in order to obtain large power. Installations must be designed to obtain maximum efficiency over the normal range of operation rather than at the maximum load. Often 2 or 3% in over-all efficiencies amount to more in the aggregate than interest and depreciation on the entire cost of machinery. There is a tendency at present to sacrifice turbine efficiency for high-speed generators in order to keep down the initial cost of machines. This may not be warranted when capital costs, operating conditions, and the amount and cost of output are considered.

It is evident that the previous discussion touches only a few of the points which must be considered in the economical design of hydro-electric developments, and that study of all the elements of design is essential for the best economy.

6.—Economical and Substantial Construction

This subject, although worthy of extended discussion, will be dismissed with only a single paragraph. The cost of construction can usually be reduced by a complete knowledge of the actual conditions. Any doubt concerning the contingencies of construction will add to the contractor's estimate of cost and, if the work is done by day labor, will make the question of construction equipment doubtful, and is likely to entail expensive delays. In the construction of hydro-electric plants, adjustments must continually be made between reasonable security and unnecessary expense, and few criteria are available outside extensive and broad experience. Safety and economy are both essential and neither should be sacrificed.

7.—Economical Financing

Uneconomical financing has been one of the greatest handicaps in the economical development of independent hydro-electric plants. Many projects have been financed at low rates of interest only by such heavy discounts on the securities that the interest charges have absorbed most of the income in excess of other fixed charges and operating costs. In this way, all possible profits, at least for a long term of years, have been absorbed in the financing.

It is comparatively easy for an operating industry that is showing fair returns on the investment, to obtain the money necessary for reasonable expansion, or for its current business. The cost of financing a corporation the property and business of which are both a question of future development, and necessarily somewhat speculative, is more expensive. A hydro-electric company can seldom obtain from its own stockholders the necessary funds to complete its project unless a market at a known price is assured, and, then, only if the project is comparatively small and high returns are believed to be certain.

If the cost of development is several million dollars, the financing of a project is possible only through the agency of investment companies. Before purchasing such securities, reputable companies will require the project to be examined by men of the highest ability and long experience, who will pass on the technical features of the construction, the expense involved, the market available, the legal features, and the probability of complete commercial success. Such companies will not assist a project of this kind without fair returns both for themselves and for their clients.

Many regulatory bodies have found it to be to the best interest of both the investing public and the stockholders of a project to limit the bonded indebtedness to about 75% of the cost, and to raise the remaining 25% by the sale of junior securities, usually in the form of preferred and common stock. The sale of the preferred stock provides the necessary margin of equity back of the bonds, and also makes the employees and the public partners in the enterprise. The writer believes that the greatest economy in financing will obtain if the bond issue is limited to about 50% of the capital cost.

The preceding method of financing, although different from that used in many of the earlier projects before State regulation was in force, is sound and will be the most economical for the industry in the long run. If, as can reasonably be expected of an economically sound project, the rates are sufficient to yield a net return of 8% on the actual cost of the property, the profits to the common stock will be attractive.

8.—Rapid and Economical Development of Business

The high cost of the development of a new business is rarely appreciated. Cost of property and construction is obvious and usually will be reasonably provided for in the estimates. It should be equally obvious that time and expense will also be required to create an independent business after the construction of the plant is completed and before a market can be developed.

FIG. 4.—VIEW OF RESERVOIR AND DAM OF CITY WATER-WORKS, OKLAHOMA, OKLA.

When the plant and equipment are first ready for operation, the business of the plant is, in general, only partly, if at all, developed. It is yet to be demonstrated whether the project was advised, designed, and constructed properly. Faults must be corrected, weaknesses remedied, machinery adjusted, a staff trained, and a business established. The plant must operate, even at a loss, if it is to fulfill the functions for which it was built. During this period, its income is commonly insufficient to meet the operating expenses, fixed charges, and fair returns. To create a business that will produce an income sufficient to meet the legitimate expenses and a fair return, will require time, and involve direct expense and losses in meeting extraordinary expenses as well as necessary fixed and operating costs, and a temporary absence of profits.

The contingencies in the development of a successful business are among the most serious factors in any undertaking. Proper engineering advice will anticipate the contingencies of construction, and experience will closely estimate costs of construction and operation. Even experience, however, can but poorly forecast the local idiosyncrasies on which the development of a business depends, unless a similar business has already been built up in the community. These facts are well known and recognized by financiers and investors and are reflected in higher rates demanded for bonds, preferred stock, and other forms of securities. After the business has been developed, when the project has become a going concern and is earning fair returns on its actual value, a far different condition obtains. The property will then have proved its fitness for service and will have justified its creation, which gives investors a confidence that at once improves the financial prospects of the venture, increases its ability to finance extensions or to re-finance its loan on a lower basis, makes it a more conservative investment, and increases its value.

The history of any business development will substantiate these statements. As an example may be cited a modest hydro-electric development in the Middle West in which real money was invested in both common and preferred stock, and the cost of financing was, therefore, moderate. The bonded debt of this company was less than half the total investment, and the preferred stock issue was about half that of the common. This company completed its plant about 1912 and was obliged, in financing, to sell its bonds on about a 7% basis and its preferred stock on an 8% basis. By 1916, it had passed the development period, which cost its stockholders in expense and deferred income more than 15% of its total capital cost. In re-financing the project during 1916, in order to provide for plant extension, the company was able to retire its old securities and place its bonds and preferred stock at 5½% and 7%, respectively. This improvement in financial conditions immediately increased the value of the common stock by an amount greater than the development expense incurred.

In order to illustrate the principles involved and the methods that may be used in the study of a problem of this kind, a brief summary of the financial aspect of a hydro-electric development proposed in one of the Western States is given, as follows:

The cost of the hydro-electric development was estimated at $9 103 000, with $7 044 000 additional for the transmission system, making a total of $16 147 000 for the entire installation. As the State was contemplating the development of this project, and the bonds would be an obligation of the State, the interest on the capital to be invested would not exceed 5 per cent. On this basis, the annual cost of power, regardless of the total output generated and sold and including depreciation, was estimated at $1 412 050. The total annual output of the plant was estimated at 87 600 000 kw-hr. per annum which would result in a unit cost of 1.6 cents per kw-hr. when all the power was sold. As one of the main purposes of this plant was to further the industrial development of the State, it was proposed to sell the output at a price as low as possible, which was estimated at 1.87 cents per kw-hr. The generating costs in the various municipalities of the State varied from 1 cent to 15 cents per kw-hr., on account of the high cost of fuel. It was assumed that, within the first year after the construction of the plant and transmission line, a total load of 30 000 000 kw-hr. per annum could be obtained, and that the annual growth would be 10 per cent. The sale of 30 000 000 kw-hr. per annum on the basis of an average selling price of 1.87 cents per kw-hr. would be insufficient to pay interest and operating expenses, and the deficit would amount to $407 500 for the first year. With a 10% annual growth in the market, the deficit would decrease each year until the sale of power reached an annual amount of 53 000 000 kw-hr., when the loss would cease, and thereafter a profit, above interest and operating charges, would result. Table 2 shows in detail the estimated annual loss and gain, considering only interest and operation costs, with the 10% annual growth as estimated. From this table it will be noted that, if the plant had started with an annual output of 30 000 000 kw-hr. in 1919, and the output had increased uniformly at an annual rate of 10%, by 1924 the total loss (not considering interest on the deficit) would have amounted to $1 486 400, and that if interest on the losses were included, the total loss would have amounted (in 1925) to $1 812 168, not including depreciation. If depreciation is included on the assumed straight-line basis (of 100% depreciation in 38 years), the total loss would be $7 458 158, or about 46% on the cost of the plant. This amount is the cost of developing the market and is a part of what is ordinarily termed development expense. This is commonly represented by lost dividends in a privately owned plant and increases until the earnings reach a normal dividend-paying basis.

With the State plant, a profit (above interest and operating expenses) is shown for 1925, which increases annually until the capacity of the plant is reached in 1931, after which the annual profit would remain essentially stationary unless the sale price is changed.

Considering the entire thirteen years represented in Table 2, the gain (over operating expenses) exceeds the losses by about $1 048 000, although if interest on the deficits is considered, this amount would be reduced to about $300 000. This would be increased thereafter at the rate of about $675 000 per annum until a normal depreciation fund is established after which the earnings could be applied to the liquidation of the capital cost.

The calculations for the following years are shown in Table 3. The interest losses would be recovered by the end of the thirteenth year (1931). The assumed depreciation would be recovered by the end of the twenty-fourth year, and if the total income above interest and operating expenses was applied to paying off the bonds, the capital cost would be paid at the end of the twenty-ninth year. For the remainder of the useful life of the plant, the savings above operating expense, interest, and depreciation would accumulate to (without interest on accumulation) $13 266 000. Fig. 5 shows the theoretical financial operation of a contemplated hydro-electric development, giving estimated annual income, operating expenses, interest and depreciation, with all moneys applied to retirement of bonds until paid and ultimate earnings before final 100% depreciation. Fig. 6 illustrates the theoretical financial operation of a contemplated hydro-electric development, showing estimated capital invested, capital cost of developing market, retirement of bonds, and net earnings before ultimate final 100% depreciation, based on the annual financial showing of Fig. 5. It is evident that the depreciation assumed in this example is much too great and that the funds would not be used as outlined. Part of the funds would be expended in maintenance and replacements. The bonds would run for a greater length of time, and the plant, if properly constructed, operated, and maintained, would still be in a satisfactory operating condition at the end of thirty-eight years.

TABLE 2.—Development Expense Based on Possible Interest Losses During Early Years of Operation; Including Cost of Entire Plant with Primary and Secondary Transmission System. Depreciation Deferred.

Year.	Output, in kilowatt-hours.	Cost of power, in cents.	Average sale price, in cents.	Difference of cost and sale price, in cents.	Loss.	Profit.	Accumulated loss, including 5% interest charge.	Accumulated profit, including 5% interest charge.
1919	30 000 000	3.23	1.87	—1.36	$407 500	.....	$ 407 500	.....
1920	33 000 000	2.94	1.87	—1.07	353 000	.....	780 875	.....
1921	36 300 000	2.67	1.87	—0.80	290 500	.....	1 110 419	.....
1922	39 930 000	2.43	1.87	—0.56	223 500	.....	1 389 440	.....
1923	43 923 000	2.21	1.87	—0.34	149 000	.....	1 607 912	.....
1924	48 315 300	2.00	1.87	—0.13	62 900	.....	1 751 207	.....
1925	53 146 830	1.82	1.87	+0.05	.....	$ 26 600	1 812 168	.....
1926	58 461 513	1.66	1.87	+0.21	.....	122 799	1 780 075	.....
1927	64 307 664	1.51	1.87	+0.36	.....	281 500	1 637 580	.....
1928	70 738 430	1.37	1.87	+0.50	.....	353 500	1 365 959	.....
1929	77 812 273	1.24	1.87	+0.63	.....	491 000	943 257	.....
1930	85 593 500	1.13	1.87	+0.74	.....	684 000	356 420	.....
1931	87 600 000	1.10	1.87	+0.77	.....	675 000	.....	.....
	Total.....				$1 486 400	$2 534 300	.....	.....
	Loss without interest.....				.....	1 486 400	.....	.....
	Profit without interest.....				.....	1 047 900	.....	$300 759*

*Profit including interest on early losses.

A further examination of Table 3 will show that if the market could be developed, so that the demand at the time of completion of the plant would

reach 53 000 000 kw-hr. per annum, the income received would meet the interest and operating expenses and would be sufficient by 1929 to meet the annual depreciation charges as well.

The rate of growth, as well as of the stability of the power market, may seriously affect the cost of developing a market. Stagnation in development, due to inability to effect sufficient sales of power, to financial panic, or to other causes, might involve a continuing loss for some years. This is a risk which the development of such a market naturally entails and which cannot be obviated.

TABLE 3.

End of year.	Earnings.	Accumulated earnings.	Interest. at 5%.	Retirement fund.	Accumulated depreciation.	Accumulated net profits.
1931	$ 675 000	$ 300 000	.....	$ 300 000	$ 5 645 990	.....
1932	"	975 000	$ 15 000	990 000	6 080 220	.....
1933	"	1 665 000	49 500	1 714 500	6 514 450	.....
1934	"	2 389 500	85 725	2 475 225	6 948 680	.....
1935	"	3 150 225	123 761	3 273 986	7 382 910	.....
1936	"	3 948 986	163 790	4 112 776	7 817 140	.....
1937	"	4 787 776	205 638	4 993 414	8 251 370	.....
1938	"	5 668 414	249 670	5 918 084	8 685 600	.....
1939	"	6 593 084	295 904	6 888 988	9 119 830	.....
1940	"	7 563 988	344 449	7 908 437	9 554 060	.....
1941	"	8 583 437	395 421	8 978 858	9 988 290	.....
1942	"	9 653 858	449 392	10 103 250	10 422 520	.....
1943	"	10 778 250	505 162	11 283 412	10 856 750	.....
1944	"	11 958 412	564 170	12 522 582	11 290 980	.....
1945	"	13 197 582	626 129	13 823 711	11 725 200	.....
1946	"	14 498 711	691 185	15 189 896	12 159 400	.....
1947	"	15 864 896	759 494	16 624 390	12 593 700	.....
1948	1 474 000	.....	.....	.....	13 027 900	$ 1 474 000
1949	.....	.....	.....	.....	13 462 100	2 948 000
1950	.....	.....	.....	.....	13 896 400	4 422 000
1951	.....	.....	.....	.....	14 330 600	5 896 000
1952	.....	.....	.....	.....	14 764 800	7 370 000
1953	.....	.....	.....	.....	15 199 000	8 844 000
1954	.....	.....	.....	.....	15 633 300	10 318 000
1955	.....	.....	.....	.....	16 067 500	11 792 000
1956	.....	.....	.....	.....	16 501 800	13 266 000

It is evident, therefore, that the success of a contemplated development depends not only on the original cost of installation, but also on the growth and development of a power market. It is not assumed that this showing would be attractive to a State, and much less to a private corporation, but it it believed that the principles involved and correctly applied furnish an idea of how such studies may be made. Any plant can then be studied on such basis as may seem desirable. The basis should be in general conservative, even to the extent of being severe. A project should show a considerable margin of safety to take care of future conditions which cannot be foreseen.

9.—Economical Management, Operation, and Maintenance

One of the weaknesses of all investments, especially those in common stock, in corporations of considerable size, is due to the fact that majority stockholders may mismanage a property and operate it improperly and extravagantly before they can be checked by legal proceedings, and even in such an event

legal relief is inadequate to protect minority interests. It is this fact that makes stock investments in anything except an established business exceedingly hazardous unless the stock purchased will be controlled by friendly majority interests. Influential stockholders occupying official positions are often able to obtain for themselves salaries incommensurate with their services and duties, and then operate industries for the benefit of themselves and their friends. The legal right is often mistaken for the moral right to obtain unjust returns to the detriment of the minority stockholders. It is unnecessary to discuss these factors at length. Honest and economical management is highly essential to the success of any project.

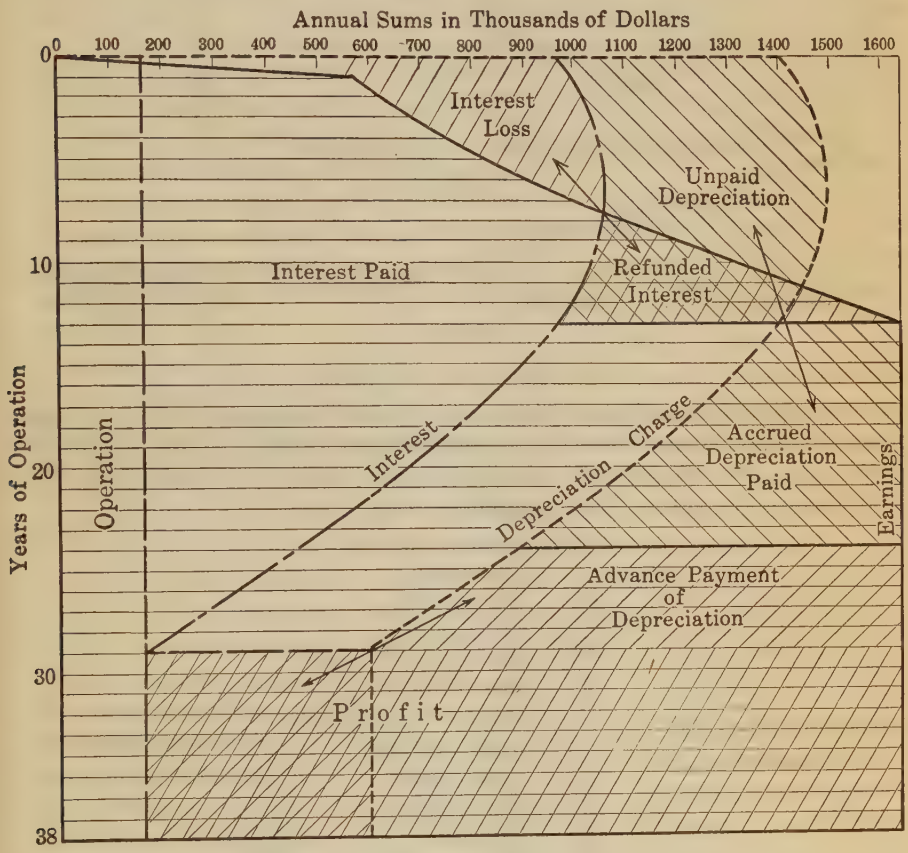

FIG. 5.

The operators of a plant should be well trained and experienced, adequate in number, but limited by the best interests of the plant. Attention should be given to every possible source of waste, and maintenance must receive adequate attention. These matters can result only from honest, intelligent, and experienced management, which is sometimes a considerable hazard in new business ventures.

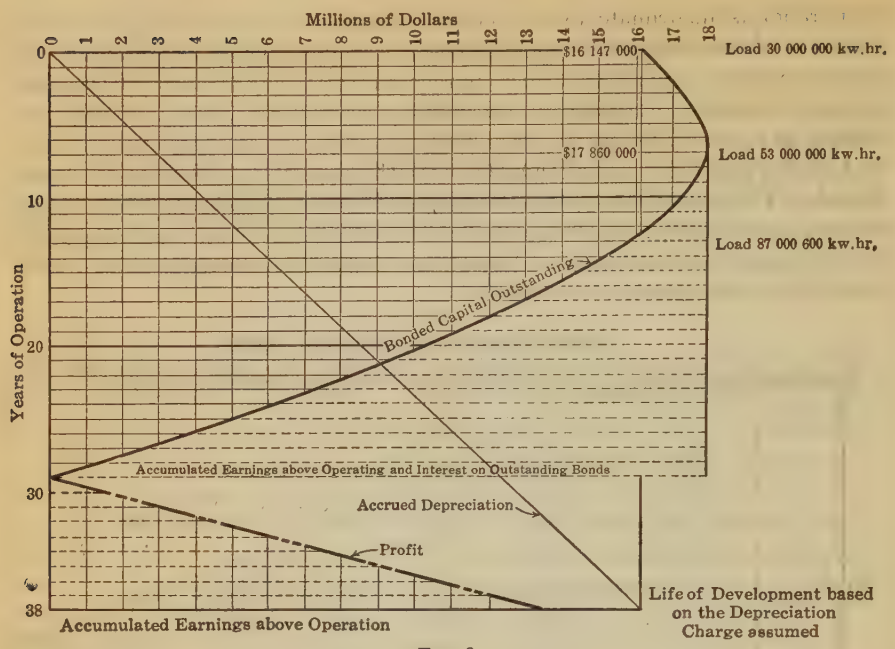
Millions of Dollars
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18
$16 147 000
Load 30 000 000 kw.hr.
$17 860 000
Load 53 000 000 kw.hr.
Load 87 000 600 kw.hr.
Years of Operation
0
10
20
30
38
Bonded Capital Outstanding
Accumulated Earnings above Operating and Interest on Outstanding Bonds
Accrued Depreciation
Profit
Accumulated Earnings above Operation
Life of Development based on the Depreciation Charge assumed

FIG. 6.

10.—ACCURATE ESTIMATES

The economic expediency of any hydro-electric development will be justified by a favorable balance between the annual income from the power sold, on the one hand, and the annual expenditures for fixed charges plus operating expenses on the other. (See Appendices B and C.)

Power sold × Price = Annual income *vs.*.....

- Interest on Cost of:
  - Promotion
  - Engineering
  - Financing
  - Rights and Property
  - Construction
  - Market Development
- + Annual Cost of:
  - Depreciation
  - Management
  - Operation
  - Maintenance
- + Annual Expense of:
  - Insurance
  - Taxes

= Total annual expense

Balance = Dividends or losses

Before a project is undertaken, reasonable assurance that this balance will be favorable should be obtained from accurate estimates of power output and

income, and of cost of promotion, financing, construction, depreciation, development of market, management, operation, and maintenance. Such estimates depend on correct and complete analyses of each of the factors heretofore discussed, and should include allowance for contingent expense proportional to the hazards involved. Optimism on the one hand and pessimism on the other must be eliminated and replaced by thorough investigation and broad experience.

The development of a hydro-electric project is not a simple and easy method of surely capitalizing the waste energy of streams and profiting from the returns. It involves many factors besides those of design and construction. Failure or neglect to consider properly and determine accurately the influence of any one of the factors previously discussed, may create serious hazards in the venture, and has resulted in failures more or less complete.

In the past there has been an idea in the minds of the public, which has been shared by many investors and by many engineers, that as undeveloped water power is energy going to waste, the development of such power must always result in large and profitable returns. On this hypothesis, and with little or no additional consideration, some hydro-electric projects have been conceived, designed, and constructed, and have resulted either in utter failure or in only partial fulfillment of the hopes that inspired their promotion. With the increasing demand for power, coupled with the popular conception that water powers were always exceedingly profitable, and that by means of such developments the waste energy of a stream could advantageously be turned into dividends, investors eagerly sought hydro-electric investments. The results have not always been satisfactory, and a considerable number of large and small developments might be cited, which have resulted in serious financial catastrophes and in serious losses to the original investors in the enterprises.

Foreclosures and sales of water-power properties have been only too common and, at present, there are some developments operating under conditions which barely permit of their survival. In one case within the State of Wisconsin, after foreclosure, the investors in the bonds of a water-power company realized less than 5% of their par value. In another case, the plant was definitely abandoned and dismantled (see Fig. 7). It is evident that such projects were ill-advised and should never have been undertaken or, if attempted, undertaken on a more conservative basis. In many of the failures which might be cited, it is evident that most of the difficulties in which hydro-electric developments have been involved, have been due to lack of foresight, as to the factors necessarily involved in a successful development, or to lack of a thorough investigation and determination of the character of these factors and their influence on the project. Most if not all these failures could probably have been avoided by intelligent and thorough investigation which would have led either to the abandonment of the project or to a more conservative development.

Plans for large and important structures are rarely devised, that do not require modification during construction. Unless this fact is appreciated by

the designer, and liberal allowance made therefor, the estimate of cost will always be inadequate to complete the project.

The hazards of construction increase with the difficulties. When a structure is built in and across a river, the work of construction is subject to certain hazards which cannot fully be foreseen, but which must be recognized and provided for in the cost estimates; there are contingencies of foundation which require careful investigation, and of flood which experience has shown to be very important. The investment is not easily determined, and the ultimate cost is sometimes greater than inexperience would deem possible.

In a recent large water-power development which was under advisement for a number of years, and which was supposed to be thoroughly considered and carefully planned both as to its design and the methods of construction, the cost of the finished structures exceeded the estimate by 33 per cent. In another case, with which less care was used, on account of the belief in the great profits from hydro-electric developments, the cost of the complete development was more than three times the original estimated cost; and since its foreclosure and sale, the plant has not been found to be a profitable investment, even on the basis of the purchase price, which was 20% of its actual cost.

The unexpected extra costs of such developments due to unforeseen delays are often serious. The interest on bonds must be met semi-annually or annually from their date of issue; hence, interest during construction is an important item in the cost of development of any industry, and an item which is particularly uncertain in water-power development. In a recent hydro-electric development, a flood (the largest that had occurred on the river within the limited period of record) not only caused a large loss to the work under construction, but was followed by continuous and unusual high water for most of the year following, so that not more than ninety working days were available within the year. In the same project, an ice jam in the spring carried away the trestle and falseworks. These misfortunes caused a delay of more than a year, with an extra interest cost of many thousands of dollars.

Such hazards obtain more in all classes of hydraulic work than in almost any other kind of development, except mining. Although care and experience with hydro-electric projects will result in more careful investigation and more liberal estimates so as to provide for contingencies that are likely to occur, still the contingencies exist and the necessary investment, in the future as in the past, will frequently be under-estimated. The costs of construction will often require greater investments than the optimistic promoters will think possible, even when money is judiciously and cautiously expended.

Even after a development is completed, all contingencies are not removed. Not many years ago, a flood in one of the rivers of Wisconsin caused a loss of several hundred thousand dollars to a single development. This loss resulted from an extraordinary and uncommon condition. The gates of the plant were clogged by an unexpected rush of logs, accidentally released from the boom that confined them. The flood-waters, unable to pass the gates, cut their way around the structure, made necessary a large expense for new con-

FIG. 7.—VIEW OF DISMANTLED AND ABANDONED HYDRO-ELECTRIC DEVELOPMENT IN WISCONSIN.

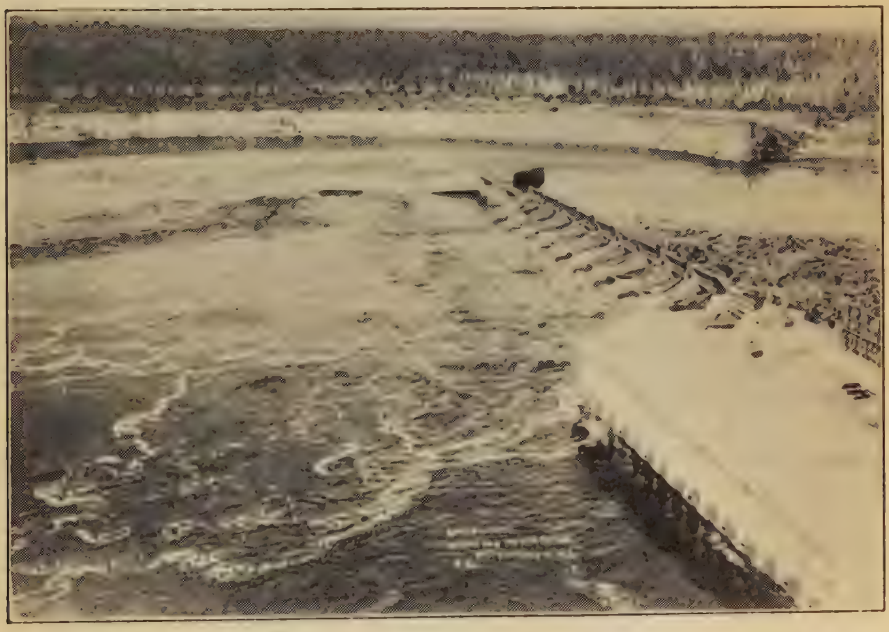

FIG. 8.—VIEW OF DAMAGE TO HYDRO-ELECTRIC DEVELOPMENT DUE TO LOG JAM.

struction, and put the plant out of service for more than a year. (Fig. 8.) At another plant development, a dam was seriously injured, and the plant was out of service for more than a year, because of a flood resulting from the destruction of another dam up stream. The injured dam possessed sufficient strength and flood capacity for all contingencies of normal flow that were likely to occur, yet the destruction of another structure and the release of thousands of acre-feet of water at the peak of a maximum flood, resulted in large expense for reconstruction and in serious loss of service.

Losses through defective designs are common. In a Western plant, the whole foundation was washed from under the dam, leaving the structure suspended from the rock walls of the narrow canyon. This loss was due to the defective design which could easily have been corrected, but it cost the company many thousands of dollars and delay of a year in beginning operations. Numerous instances could be given in which either fundamental defects or extraordinary conditions have destroyed or seriously injured dams and hydro-electric plants and caused serious losses to their owners. Knowledge of the possible activities of natural forces is limited; the effect of the possible combination of the known and unknown factors can never be clearly seen, yet these contingencies are ever present and must be considered by the water-power designer and investor, and contingent costs must be allowed in estimates in proportion to the hazards involved.

In estimating power, income, and costs in hydro-electric developments, it is evident, therefore, that a thorough study of each factor is essential to a correct solution of the problem.

Before closing it seems pertinent to discuss briefly a still broader field of the economics of hydro-electric developments; that is, the relation of such development to the public, the State, and the Nation. Much has been publicly said and written adverse to the development of these powers by private interests and favorable to State or Federal ownership and development. In the first place, the great value of the majority of the water powers of the Nation is potential only. More than 50% of these powers could not profitably be developed, and they are, therefore, valueless except on the speculative basis of future possibilities.

The development of these powers adds greatly to the tax income of the State and the Nation. Their utilization entails extensive developments in industry, in opportunity, in population, and in the value of taxable property. These powers decrease the demand for coal from 5 to 10 tons per annum per horse-power developed, which is equivalent to a saving of from one-sixth to one-third of a carload of coal per annum per horse-power. This transporting capacity can then be devoted to the service of an equivalent amount of manufactured or agricultural products. The development of these powers entails an equivalent development in industry and an equivalent satisfaction of some service or convenience to the people of the Nation. The State in which they are developed generally receives in taxes about 2% per annum of the cost of the development per annum which, within the 50-year limits of a Federal franchise, amounts to as much as the capital cost of the development, and the indirect income from industrial development will be fully equal and probably greater.

The benefit to the Nation, the State, and the community is such that, regardless of the trend of future ownership, the benefits derived by the public warrant every reasonable encouragement by the State and Federal Governments.

Although these developments are directly undertaken for the possible profits to be derived from their construction and operation, the public receives, in general, no less benefit from their development than the corporations which develop them; and, in some cases, it can be shown that within ten years of the construction of such developments, their customers have received in reduced power charges more than the capital costs of the entire development. Direct benefit to the people is assured from the very nature of the commercial conditions of the sale of such power, as has already been pointed out, and even under non-competitive or monopolistic conditions, their benefit is assured through the State control of power rates.

It is true that the public often anticipates greater saving through rate reduction than commonly is possible. Distribution cost is usually about 60% or more of the total cost of delivered electric power, therefore, any cheapening in cost of power generation will not effect more than 40% of the cost to the consumer.

In supplying power to rural lines, the distribution cost is commonly more than 90% of the total cost; and even if the generation of power cost nothing, the price of power from such lines will necessarily be high, compared with the price at which it can be furnished in cities and especially to large consumers.

Incidentally, it may be stated that the popular and newspaper conception of interconnection of plants and "super-power" developments is often erroneous. The mere interconnection of plants may not of itself effect economies or in any way warrant a reduction in power rates. Such interconnections may be, and actually are, warranted in certain cases, and frequently provide greater security and stability in the power supply of any community by affording an additional source of supply in case of local accident, or even due to local low-water conditions, and, broadly speaking, such interconnection may bring about certain economies that may ultimately result in rate reduction. Such connections, however, cannot produce radical reduction in rates, or bring to small customers on rural lines the advantage of the energy rates of large customers, as is sometimes apparently anticipated.

Those who are developing the resources of the Nation under reasonable conditions are not enemies of the public, and the writer believes that he who causes two blades of grass to grow where one grew before is no less worthy of credit if he garners his harvest and sells it for his own profit.

APPENDIX A

Factors of Economic Expediency in the Consideration of Proposed Hydro-Electric Developments

The success of hydro-electric developments will depend on the following factors:

1.—The acquisition of suitable legal rights:

(*a*) To a franchise or legal right to build and operate.
(*b*) To the water supply.
(*c*) To the site for dam and power station.
(*d*) To the flowage, including pondage and storage.
(*e*) To the rights of way for ingress, egress, transmission lines, canals, pipe lines, etc.

2.—The command of a satisfactory market:

(*a*) Within reasonable transmission distance.
(*b*) Of sufficient size to utilize a large proportion of the power.
(*c*) Of a character to permit of an economic development (high load factor).
(*d*) At a price sufficient to warrant the investment.
(*e*) Of a stability that will warrant a permanent or definitely limited investment in real estate, water right, plant, transmission line, etc.
(*f*) Reasonably free from likelihood of damaging competition.

3.—Reasonably suitable physical (topographical and geological) conditions:

(*a*) For a dam site and sites for construction of power-house and appurtenances.
(*b*) For the development and maintenance of a suitable head.
(*c*) For stable and permanent structures at reasonable expense.
(*d*) For impounding water for pondage and storage, with only moderate losses from percolation and evaporation.
(*e*) For economical conduct of the work:
For transportation of materials and machinery.
For securing labor, teams, and construction equipment.
For securing bidders on the work.

4.—Favorable hydrological conditions:

(*a*) To provide a stream flow not subject to extreme variations of discharge; that is fairly well maintained at low-water seasons, and free from extreme floods.
(*b*) To require only a small proportion of auxiliary power.
(*c*) To provide the amount of power necessary for the market, with the use of such other water or auxiliary power as is economically available.

5.—Proper design:

(*a*) Location selected must be the most favorable:
To afford suitable foundation for dam, power-house, and other work.
To afford ingress and egress and reasonable transportation facilities.
To afford sufficient pondage to equalize the flow and permit its utilization on the load factor under which it will operate up to the point of economical development.

(*b*) The dam:
Must have suitable foundation.
Must have ample capacity for maximum flood.
Should have gates to keep low-water head at maximum while providing ample capacity (with gates open) to keep head and back-waters at their designed limits.
Must be designed for strength and stability; also, to resist shocks of floating ice or débris and for ice pressure.

Must have base and ends essentially tight against seepage.
Must be designed to prevent overflow around ends.
Must be designed to pass ice, débris, and logs.
Must be designed with suitable provision against erosion down stream from the structure.
Should have such logways, fishways, mud valves, sluice-gates, or head-gates, as may be necessary.
Should be long enough for passing floods and short enough to avoid high cost.

(*c*) Head-race (where needed):
Should have economical section to prevent loss of head (and energy) under normal flow, at peak load, and under ice conditions.
Should be provided with a forebay just above the plant for small storage to permit change of load without serious loss of head.
Usually with head-gates or stop-log openings.
Usually with booms to prevent ingress of floating débris.
Lined to prevent seepage and to reduce friction.
Stable to reduce cost of maintenance and repairs.

(*d*) Pipe penstocks or tunnels (where needed):
Economical section.
Proper head-gates and protection from drift.
Proper inlet gates for filling penstocks slowly and equalizing pressure on head-gates before opening.
Air inlet to prevent collapse when being emptied.
Stand-pipe or surge tank to prevent shocks and stabilize running condition of turbines and generators.
Proper entrance to turbine chamber.

(*e*) Power-house should be designed:
As to substructure:
On safe and stable foundation.
With trash-racks to keep débris from turbines, but with spacing and section not to interfere with flow and suitable for convenient and easy raking.
With trash-gates to permit disposal of trash.
With head-gates to cut off supply from turbines and inlet gates to fill penstocks and equalize water pressure.
With suitably shaped inlet passages to turbines.
With suitable draft-tubes and discharge sections to reduce velocity losses.
As to superstructure and sub-stations:
With ample room for equipment.
Arranged with regard to safety in operation.
Arranged to facilitate operation (so that the right way is the most convenient and easiest way).
For future enlargement or extension (if probable).
Fire-proof, with ample light and suitable ventilation.

(*f*) Tail-race:
With economic section to facilitate discharge and economize head (see also "Head-Race").

(*g*) Equipment:
General:
Gate-operating machinery for ease and rapidity of handling gates.

Cranes for economic erection and maintenance of machines.
Recording gauges to give suitable records.
Turbines:
Of an economical type.
Well-designed and constructed.
Equipped for renewable parts.
Not too large for economical operation.
With fly-wheels, if necessary for proper regulation.
With sensitive governors.
Electrical equipment, generators, exciters, switchboard, protective devices, automatic or remote control, lightning arresters, transformers, oil purifiers, switching, and metering equipment.

(*h*) Transmission line:
Right of way easy of access for construction and maintenance, ample in width to protect line from falling timber, encroachments, and fires.
Safety to public by warnings, height of lines, and protection under lines at highways, railroads, and other crossings.
Economical voltage for distance.
Conductors large enough for proper regulation and low-power loss.
Adequate insulation and lightning protection.
Mechanical strength to withstand wind and sleet.

6.—Economical construction:

(*a*) Proper supervision, inspection, testing, etc.
(*b*) Foresight and provision for contingencies.
(*c*) Judicious contracts at reasonable prices, or construction by day's work under experienced economic supervision.
(*d*) Judicious purchase of materials, machinery, equipment, and supplies.
(*e*) Curtailment of undue salaries and general expenses.

7.—Financing:

Although this item is included under "Accurate Estimates", it also requires special consideration, as many developments are seriously handicapped by under or over-financing, or by too great cost of financing.

8.—Rapid and economical development of business:

Although this item is considered under "Command of a Satisfactory Power Market", attention is especially called to the fact that the amount of power which can be sold immediately, and the price which can be obtained, is frequently over-estimated; also that the time required for developing a profitable market often requires a larger extra investment in actual cost, interest on bonded debt, and deferred dividends.

9.—Economical management, operation, and maintenance:

A development otherwise satisfactory may be seriously handicapped through unwise and extravagant management. Honest and economical management is highly essential to economic success. Experienced and trustworthy operation and maintenance is also essential. Waste, with consequent loss of output, may arise from neglect in operation, such as racks clogged with débris, leaking turbine gates or runners, and the like.

Many such sources of loss and waste are possible and can be obviated by wise and intelligent work.

10.—Accurate estimates of power output and income, and cost of promotion, financing, construction, depreciation, development of a market, management, operation, and maintenance:

The final criterion of economic development of any hydro-electric power is a favorable balance between annual income and annual expense. An accurate forecast of all the items which go to make up this income and these expenditures is, perhaps, the most difficult in an investigation, and the balance should be sufficiently favorable to cover all hazards that are likely to occur.

APPENDIX B

Capital Costs of Hydro-Electric Developments

1.—Preliminary Investigation Expenses:

(*a*) Legal investigations of water rights, land rights, transmission rights of way, etc.
(*b*) Surveys, geological and hydrological investigations.
(*c*) Foundation conditions, borings, etc.
(*d*) Census of power market and determinations of stability and sale price of power.
(*e*) Preliminary designs and cost estimates.

2.—Promotion and Organization Expense:

(*a*) Promotion costs and profits:
(*b*) Legal expenses of organization.
(*c*) Cost of obtaining franchise.
(*d*) General expense of organization.
(*e*) Expense of financing and placing stocks and bonds or other securities.

3.—Real Estate and Water-Right Expenses:

(*a*) Sites for dam and power-house.
(*b*) Flowage and water rights.
(*c*) Rights of way for canals, roads, and transmission lines.
(*d*) Sites for auxiliary plant and sub-station.

4.—Cost of Plans:

(*a*) Final and complete surveys and investigations of site, flowage, and location of canals, roads, and transmission lines.
(*b*) Complete plans, specifications, and contracts.
(*c*) Final estimates of cost of complete plant and equipment.

5.—Construction Costs:

(*a*) Cost of clearing lands, including flowage.
(*b*) Cost of plant with complete equipment, including materials, labor, construction equipment, and plant supplies.
(*c*) Cost of operators' houses, if required.
(*d*) Contingent cost, with due regard to each feature of the development.
(*e*) Cost of engineering, supervision, inspection, tests, administration expense, including salaries, insurance, office equipment, and office supplies.

6.—General Costs:

(*a*) Insurance and taxes.
(*b*) Interest during construction.

(*c*) Interest during development and development expenses.
(*d*) Stores and supplies.
(*e*) Working capital.
(*f*) General administration expenses.
(*g*) Expense of financing (if not included in promotion expenses).

APPENDIX C

Annual Expenses

1.—Salaries and Wages:
Officers, bookkeepers, and clerks.
Management and supervision.
Power salesmen and adjusters.
Operators, linemen, etc.

2.—Supplies:
Fuel and oil.

3.—Maintenance:
Repairs and replacements.

4.—Interest:
On bonds, notes, and floating debt.

5.—Discount on:
Stock and bonds, when not made a capital charge, must be provided for by an annual charge.

6.—Development Cost:
When not provided for as a capital charge, this must be absorbed as an operating expense.

7.—Replacement Reserve:
To provide for unusual damages to plant and replacement of major units of plant.

8.—Sinking Fund:
To retire values when values may be destroyed under a limited franchise, and to reimburse the investor on the expiration of the franchise.

APPENDIX D

Calculation of Net Revenue (Gross Revenue Less Steam Power Cost) for Peninsular Power Company

By F. E. Turneaure, M. Am. Soc. C. E.

The lower half of the average duration curve for the Twin Falls Plant is closely expressed by the equation:

$$y = 1\,800 + \frac{t}{6}(2\,200); \text{ or } t = \frac{6}{2\,200}(y - 1\,800)$$

in which,

y = horse-power available from hydraulic plant; and

t = time, in months, during which the stream flow is below that necessary to deliver y h.p.

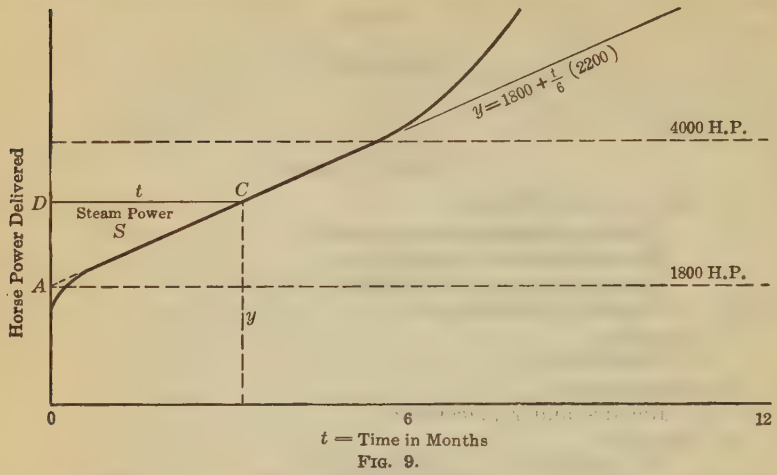

FIG. 9.

The amount of power necessary to be generated by steam in the average year, in order to supply y h.p. continuously, will then be the area, $A\ C\ D$. Expressed in horse-power-months, this equation is:

$$S = \frac{1}{2}(y - 1\,800)\,t = \frac{3}{2\,200}(y - 1\,800)^2$$

If r = average gross income per horse-power-month and c = cost of steam power per horse-power-month, the net yearly revenue for y continuous horse-power will be:

$$R = 12\,r\,y - \frac{3}{2\,200}\,c\,(y - 1\,800)^2$$

The average monthly revenue per horse-power is almost exactly \$6.00 (= 1.1 cent per kw-hr.), or r = \$6.00.

For various values of c, the amount of net revenue is, therefore, as follows:

$c = 1\frac{3}{4}$ cents per kw-hr. = \$9.65 per horse-power-month.
$R = 72\,y - 0.013\,(y - 1\,800)^2$

$c = 2$ cents per kw-hr. = \$11.00 per horse-power-month.
$R = 72\,y - 0.015\,(y - 1\,800)^2$

$c = 2\frac{1}{4}$ cents per kw-hr. = \$12.40 per horse-power-month
$R = 72\,y - 0.017\,(y - 1\,800)^2$

$c = 2\frac{1}{2}$ cents per kw-hr. = \$13.70 per horse-power-month.
$R = 72\,y - 0.019\,(y - 1\,800)^2$

For the maximum value of R, with $c = 1\frac{3}{4}$ cents, by differentiation,

$$y = 1\,800 + \frac{72}{2 \times 0.013}.$$

For $c = 1\frac{3}{4}$ cents $y = 4\,500$ h.p.
$c = 2$ cents $y = 4\,200$ h.p.
$c = 2\frac{1}{4}$ cents $y = 3\,900$ h.p.
$c = 2\frac{1}{2}$ cents $y = 3\,700$ h.p.

The capacity of the Twin Falls and the Brule Plants may be placed at about 3 900 h.p. and the steam plant will have sufficient capacity to maintain this output at low water. The capacity is, therefore, not sufficient to secure the maximum revenue when c is less than $2\frac{1}{2}$ cents per kw-hr.

The net revenue for $y = 3\,900$ h.p., for various values of c, is as follows:

For $c = 1\frac{3}{4}$ cents $R = \$223\,600$
$c = 2$ cents $R = 214\,800$
$c = 2\frac{1}{4}$ cents $R = 206\,000$
$c = 2\frac{1}{2}$ cents $R = 197\,200$

For $c = 2\frac{1}{2}$ cents and $y = 3\,700$ h.p. (the most economical value), $R = \$197\,800$, only slightly greater than for $y = 3\,900$.

If the capacity was sufficient, the value of R, for $c = 1\frac{3}{4}$ cents and $y = 4\,500$, would be \$229 000, a difference of only \$5 400 from the previously given value of \$223 600 for $y = 3\,900$ h.p.

With coal at \$10 per ton, the cost of steam power will be about $2\frac{1}{4}$ cents per kw-hr., and the most profitable output would be about 3 900 h.p., or 7 800 h.p. from the two plants with a net revenue of about \$412 000. Adding the Pine River Development at 60% of each of the others, brings the most profitable output to $3\,900 \times 2.6 = 10\,200$ h.p. and the net revenue to \$535 000.

To deliver this amount of power would require further additions to the steam plant. Without such additions, the capacity will be about 8 800 h.p., taking the low-water capacity of the Pine River at 1 000 h.p. Distributing this load proportionately will call for 3 400 h.p. from Twin Falls, giving a net income ($c = 2\frac{1}{4}$ cents) of \$201 300, or for the three plants of \$524 000, or only \$11 000 less than the maximum to be secured by additional steam. The additional steam capacity to supplement the Pine River development would be about 1 400 h.p., costing at least \$125 000. Fixed charges on this would be about 11%, or \$14 000, which shows that, at the present rates for current and for coal, there would be no profit in developing the capacity of Pine River beyond 1 000 h.p.

Query: Are the rates for power what they should be in the interests of "conservation"?

Calculation for Minimum Year (Envelope Curve for 1903-1918 Inclusive)

The same method of analysis applied to the year composed of the lowest points for the years 1903-1918, gives the following value for net revenue:

$$R = 72\,y - \frac{3}{1\,100}\,c\,(y - 1\,600)^2$$

with $c = 2\frac{1}{4}$ cents per kw-hr., the net revenue from the two plants would be as follows:

For a total of 5 000 h.p. (the present load) $R = \$306\,000$
" " " " 6 000 h.p. " " " $R = 300\,000$

The most favorable load would be 5 300 h.p. A net revenue of $300 000 would leave about $70 000 for common stock. For the three plants, the net revenue would be:

For a total of 5 000 h.p. $R = \$377\,000$
" " " " 6 000 h.p. $R = 393\,000$

The most favorable load would be 6 900 h.p. and would produce a net revenue of $400 000. For the three plants, the fixed charges, operating expenses, and dividends on preferred stock would require about $300 000 per year

DISCUSSION

L. F. HARZA,* M. AM. SOC. C. E. (by letter).—The author has emphasized the economics of the isolated hydro-electric project designed to serve a given power market with such auxiliary steam service as is needed to insure continuity and reliability of service. The opportunities for the development of isolated projects to serve a given market are fast disappearing; interconnection of power systems, even of competing companies, is coming rapidly and inevitably.

In certain localities—such as the Niagara District, the area served by the Southern Power Company, the Pacific Coast District, the mining districts of Northern Michigan and others—hydraulic power as a source of energy predominates and steam power serves as auxiliary. As such systems become larger the problem of what output can be economically derived from a given hydro-electric project becomes more and more a question of how the flow of the stream will blend with that of others already developed and feeding into the system.

The writer has in mind a property in the Middle West where several hydro-electric projects serve a mining district. All the streams are in flood at the same time and the heads of the several projects are high enough so that the flood season is the period of greatest capacity. A new project was devised to feed into this system, but little value could be given to secondary energy without a definite contract for the purchase of this energy, for the reason that this and all the previously existing hydro-electric projects feeding into the system offered surplus power during the same season. The new project, therefore, had to be predicated largely on its low-flow capacity.

There is a situation on the Pacific Coast which is of National importance as to the ultimate future. California is developing its hydro-electric power in rapid strides and is dependent largely on storage during the dry months in the summer and fall. In the Pacific Northwest States similar stream characteristics exist, although development is not taking place so rapidly. On the Columbia River in Oregon and Washington, there are large available power sites. This river is remarkably well adapted for supplementing the existing projects of the Pacific Northwest and of California also, if the economy of long-distance transmission ultimately brings this market within its reach. Its sites offer secondary power during the summer and fall when the smaller streams of the region are very low.

The Columbia River is of such large capacity as to be able to supplement the smaller powers of a large region. If the prime capacity of the Columbia River site is taken as 500 000 h.p., then there would be an available surplus from about July 1 until January 1, varying from 0 to about 500 000 h.p. and back again. This period of surplus power of the Columbia closely fits the deficiency period of the other streams.

This is one of the best illustrations known to the writer of the ultimate possibilities of blending the capacity of streams of different characteristics

* Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.

in the same region to provide economical power far in excess of what any one stream could justify if developed independently.

In the Mississippi Valley region, where population is dense and industry highly developed, heads are low, often disappearing in time of flood, and the total of hydraulic power possibilities is small in proportion to the developed steam capacity of the region. In such districts a hydraulic development must often find justification purely from the standpoint of its saving in coal for that portion of the year when the power is available.

The tendency toward high capital cost of low-head developments, together with the seasonal characteristics of some of the low-head streams, might seem to preclude hydro-electric development in this region. The fact is, however, that because of the high industrial development from steam power, an unlimited market often exists for the hydro-electric energy which can be furnished when and if available. This justifies a development of the projects to a point much further in excess of the minimum flow capacity and thereby makes available a larger amount of energy from the project than would be justified in a region dependent largely on hydro-electric sources from rivers of similar type. The fact is that many of the small and low-head developments of the Middle West are thus fully justifying themselves and proving economically profitable.

The advent of the automatic hydro-electric station has served to warrant the economical use of many small power sites which would have been entirely unprofitable under manual operation. In some of the smallest of these developments the elimination of the cost of attendance has saved as much as 6% or 8% per annum on the capital cost, thus carrying the interest charge. The saving expressed in the percentage return on capital investment by virtue of automatic operation, varies almost inversely with the size of the station—it is of small importance to a large station, while it may be the saving which justifies the development of a small station.

The writer's experience in the development of several automatic stations and his observation of subsequent results, convinces him that the small automatic station will be one of the big fields of development of the future. To each big power site that attracts the attention of the utility company there are many small ones available but largely ignored to date.

Harry A. Hageman,* M. Am. Soc. C. E. (by letter).—This paper furnishes a broad and concise outline of the many complex and often inter-related factors to be considered in determining the economics of a proposed hydro-electric development. Many of these factors have been discussed in detail in various engineering books and disconnected articles; it is a great boon to the engineer to have the problem completely outlined as in this paper.

The writer believes that it is worth while to emphasize a few of the topics about which comparatively little has been written.

Theoretically, the complete development of a hydro-electric project should utilize all the energy at the site that can economically be controlled and marketed. The exact determination of the proper capacity is, however, extremely difficult and complex on account of the many factors involved.

* Hydr. Engr., Stone & Webster, Inc., Boston, Mass.

A hydro-electric development is not different in principle from any commercial manufacturing business. The problems may be classified as (1) those pertaining to the supply of the raw material, or water supply; (2) those pertaining to the process of manufacturing, or the economical design of the plant; and (3) those pertaining to the disposal of the manufactured product, or the market. These general groups can be further sub-divided into the many factors mentioned by the author. All three are equally important, but either because of lack of experience or because precise engineering data may be difficult to obtain, inadequate consideration often is given to at least one of these general groups.

The initial capacity for a development is sometimes fixed by the immediate power demand with some allowance for growth, and sometimes largely by financial limitations. The total cost of the initial installation is often affected greatly by the layout requirements of the ultimate developments, and by the inclusion in the initial development of certain items of construction not required immediately. Because of the rapid changes in the art, only so much of the ultimate plant should be provided for as may be absolutely necessary to allow for future expansion at a minimum of expense. The details of the design and construction of later extensions should be left for future consideration.

If the proposed plant is to be one unit of a large power system, rather than isolated, the problem is further complicated. In the former case, the object is to increase the capacity and output of the system to a maximum amount at a minimum cost within the limitations of the market. In some cases the object may be to replace more expensive sources of energy or to insure continuity of service. In the case of the isolated plant, the object is to supply a pre-determined market, with proper allowances for growth during a reasonable number of years, at a price which would be attractive to prospective customers and still at a profit which would seem ample to the owners.

The success of the project is obviously dependent on the extent to which its output can be marketed. The demand for power usually varies with the hour, the day, and the season, and must be met immediately. As the supply of power is dependent on the supply of water, it is necessary either to have a natural stream flow adequate for all demands, or to regulate this stream flow to the extent necessary that it may furnish an ample supply of power when the power is required. In other words, the important problem is to market the greatest possible proportion of the total available energy, and at the same time to insure a generating capacity equal at all times to the probable load requirements.

It is best to deal separately with the problems of demand and supply. Whenever practicable the simplest method is to determine first, the exact character of the load to be carried; and, second, the means of securing the required power. When the load is nearly constant it is necessary either to secure a uniform stream flow or to equalize a variable flow by the use of storage. When the load varies, the stream flow must be either partly wasted, or controlled and regulated in accordance with the variations in load.

The term "load factor" conveniently expresses the ratio of the average load to the maximum for any given period of time such as a day, week, month, or year. It should be remembered, however, that this factor does not define the shape of the load curve, which becomes an important factor in a study of the amount of pondage required. A load that has a very sharp peak but is fairly constant for the remainder of the day will require only a fraction of the pondage necessary for one that has the same peak for several hours but is negligible at other times.

The character of the load decides the form of the load curve, which should be determined as accurately as possible, for both the present and near future. In general, its use will give more accurate results than that of percentage load factors. On the other hand, some investigations do not warrant more than broad assumptions of probable load factors.

There is a tendency at most power plants for the load to repeat the same seasonal cycles so far as the shape of the load curves is concerned, except that an increase in the load will raise the curve. For this reason it is frequently convenient to plot load curves in terms of the percentage of the peak load rather than in terms of the actual power.

In meeting commercial requirements, it is the usual practice, after estimating the probable load curves or percentage load factors, to determine how these requirements can best be obtained with the stream flow available. The isolated plant presents quite different problems from those of a plant combined in a system of other water or steam plants; it is relatively simple, especially when the minimum stream flow can be definitely determined. There is then available a certain amount of dependable continuous power, known as primary power, which can be readily estimated after taking into account the amounts of pondage and storage available. In addition, during periods of ample stream flow, there may be excess power, variable in amount and largely uncontrollable, known as secondary power.

Where the proposed plant is to be part of a large power system, special studies are required to determine the most economical combinations. Generally speaking, a plant having large storage should be operated as a peak-load plant during periods of low water and as a base-load plant during periods of ample water. Where several plants in a system have considerable pondage and storage available, it may be necessary for the new plant to operate on an intermediate part of the load curve, being neither a base nor a peak-load plant, with the result that there may be some unavoidable waste of water which would not occur if it were an isolated plant. It is sometimes possible to secure diversity of stream flow and to combine two hydro-electric plants having low flow periods at different times of the year, or having characteristics which permit the power from one to supplement that from the other.

A combination that is often particularly effective can be obtained by connecting a low-head plant on a large river, with a high-head storage development on a tributary entering the main stream above. By this arrangement the bulk of the load during periods of high water is carried by the low-head plant, while water is being stored in the reservoir of the plant above. During low water the situation is reversed, a greater load being generated by

the high-head plant using storage water, which also serves in a lesser degree to supplement the output of the plant below.

Whether the plant is to operate as a base or a peak-load plant materially affects both the total capacity and the size and number of units to be installed. If, in the future, the proposed plant is likely to be changed from one type to the other, the initial development should be designed and constructed so that the necessary plant alterations can be made without serious changes in existing structures.

In designing the initial plant, it may be that provision for future extensions would require expenditures that would make the first installation uneconomical, in which case it is proper to charge future additions with the increased expenditures rather than to make provision for them at the beginning.

The initial construction should always allow the greatest possible latitude for future extensions, as the details of the load and the design may so change between construction periods that units of different size and type may be more economical for the later installations.

If a power market must be created, or developed to use the output from a proposed water power, the venture is quite speculative, although careful engineering and financial studies will materially reduce the hazard. The same thing may be said as to the many physical difficulties to be reckoned with, such as the interference with the continuity of the power supply by trash, silt, logs, and ice; or the enforced release and perhaps waste of water for navigation or log driving; or the interference from log or ice jams, frazil and anchor ice; or even the winter freeze. Similarly, the contingency of labor strikes either during construction, or later in industries using the power, must be considered in order properly to minimize the hazards.

The past fifteen years have been years of recession in the cycles of rainfall and consequently of run-off. It has sometimes happened in a cycle of decreasing yield that the stream flow has been lower after the completion of a plant than short-term records have predicted. This, of course, results in a lower output of power and energy and a longer time to fill ponds or reservoirs than was estimated, especially if water must be released on account of other plants down stream the supply of which cannot be entirely cut off.

In closing, attention is called to the importance of permitting the engineer to follow the operating conditions at the plant, both for the purpose of allowing him to check his power estimates or to improve on his design in future plants, and for the purpose of insuring that the plant is being operated efficiently—a condition that is not always attained and for which the engineer is unjustly criticized. It is sometimes advisable for the engineer to prepare operating curves and tables which will permit the operator of the plant to ascertain readily the best combinations of units and equipment for minimizing power losses. Certainly operating statistics are to be recommended, even if some expense is involved, in order to keep the proper control of such matters, as they can always be justified when plant extensions or improvements come up for consideration.

C. P. DUNN,* M. AM. SOC. C. E. (by letter).—The writer has been impressed by the importance of the fundamental ideas set forth by Professor Mead, and believes that every engineer connected with hydro-electric design, even though his particular duties may appear to him to be purely technical, would do well to familiarize himself with the economic phases of the problem as outlined in this paper. Questions of cost and earning power are so interwoven with the scientific phases of design and construction that it is impractical to attempt to separate them in any study or discussion.

At present, men with a clear understanding of the technique of design and construction have an opportunity to broaden their activities by familiarizing themselves with the business problems encountered in building and operating electric utilities.

While the author's paper seems to follow, in general, the idea of studying an independent development, the principles enumerated are, of course, applicable to the much more common problem, the extension of an existing system.

The writer wishes to discuss certain points mentioned in the paper that deserve greater emphasis. The engineer often fails to make a dependable cost estimate, due to under-estimating the hazards of construction, to overlooking many small items which total up to large sums, to under-estimating overhead and general expenses, and, in short, to a lack of appreciation of the problem before him.

The engineer undertaking to estimate the cost of a plant amounting to millions of dollars is taking on himself a grave responsibility; the decision to build or not to build may be based on his recommendation, and the efforts of hundreds of men for several years may be exerted wisely or foolishly, depending on the clearness of his vision. He must put out of his mind the desire (which is ingrained in every engineer) to see the project with which he is connected under construction, and must weigh the problems before him with unbiased judgment.

It too often happens that the theoretical calculation of the kilowatt-hours that a proposed hydro-plant may generate, is allowed to intrude in an engineer's report without sufficient study to determine whether or not all these kilowatt-hours are usable within the limits of the daily and annual load curves of the system to which the plant is connected. A careful examination sometimes discloses the fact that not more than 60 or 70% of the kilowatt-hours originally thought available are actually usable or salable.

The forecasting of future daily and seasonal variations in the demand for power, and the study of characteristics of the forecasted curves in connection with the power available at the plant, are very important. The value of a kilowatt-hour lies not in the mere fact that it can be called a kilowatt-hour; its value is measured by whether or not it can be delivered on demand at any hour of the day or at any season of the year. A surplus hydro-kilowatt-hour at 2:00 A. M. in the wet season of the year may be compared to a cake of ice in midwinter—its value on the market is zero.

* Designing Engr., Portland Elec. Power Co., Portland, Ore.

The foregoing emphasizes the difference in value between the power generated by plants which depend on stream flow as it comes, and those which are more fortunately situated in regard to daily pondage or seasonal storage.

When there is a choice between two or more sites for a hydro-electric development, the needs of the system to which the plant is to be connected for base power or for peak power, will have a bearing on the economic value of the various sites with respect to that particular system, and will influence the choice. In general, those sites which require the construction of long conduits under high heads are better suited to development as "base load" or continuous 24-hour per day plants, and those sites which permit the location of the power house very close to the dam are most suitable for "over-development" or "peak-load" plants, providing there is sufficient pondage available to cover the daily load fluctuations on such a plant.

It is very important that stream-flow measurements be made at a contemplated site over as long a period as possible. At times, power companies find themselves compelled to make developments on streams where gaugings have been carried on for only short periods and little is known of the characteristics of the stream. An erroneous estimate of the dependable stream flow may be very costly.

Other things being equal, several small steps in development, made as the demand for power requires, are better than one large step. This is because of the fixed charges on the large step during the long period in which it is only partly loaded. Fixed charges during the "development" or loading period may easily add 50% to the cost of a plant. The writer has pointed out a graphic method of presenting this phase of the problem.*

The solution of the economic size of a development as presented in the paper is applicable only to a specific case. In the solution it appears to be assumed (1) that the demand for power will happen to be exactly the same as the amount of power that will be delivered by the most efficient combination of steam and hydro-electric plants; (2) that the load will not grow; and (3) that there are no extraneous conditions influencing the market value of power.

The problem of determining the proper size of a development is generally very complicated, and is influenced by many things outside the plant under consideration.

With the exception of chemical and metallurgical works, the cost of power is a comparatively small percentage of the expense of running an industrial enterprise. Such things as labor market, availability of materials, and transportation, have much more to do with the location of an industry than the cost of power. This being the case, it may be said that, within reason, power is worth just what it costs to get it, no more and no less.

A hydro-electric site should be developed to the point where it is most efficient economically from the standpoint of the cost of power to the entire community within transmission distance. This involves a study of its relation in operation to other sites which will be developed in the future, and of the possibility of interconnection with other systems. The study of the power

* *Electrical World*, Vol. 81, 1923, p. 219.

development program of the entire district within transmission distance may lead to a conclusion quite different from that of a study merely to determine the most efficient combination of a given hydro-electric site with steam development. It is an economic mistake to take the very cheapest power in a way that damages or destroys possibilities for additional development at a slightly higher figure.

CHARLES B. HAWLEY,* M. AM. SOC. C. E.—Popular interest in hydro-electric development is widespread. Undoubtedly a part of this interest is due to the hope that water-power construction will soon make possible substantially lower prices for electric service, but this, of course, is a vain hope because the actual cost per kilowatt-hour at the switch-board, regardless of whether it is generated by steam or water, is always small in comparison with the cost of distributing this kilowatt-hour to the average consumer. For the most part, however, popular interest in water power comes from a realization that present fuel supplies are not without limit. Notwithstanding all the attention now being directed toward hydro-electric development, it is a fact that the aggregate capacity of all the steam-power plants being built throughout the country is several times that of all the water-power projects under actual construction.

Evidently there must be numerous reasons why this is so. Some of them have already been suggested in previous discussions; the speaker will review a few of these reasons very briefly, as they have a direct bearing on the present economic and political phases of this general subject.

First.—The preliminary negotiations in connection with the promotion of hydro-electric developments require many times as much effort and patience as in the case of a steam-power project. In the first place it is usually necessary to secure the permission of both State and Federal authorities. Thorough engineering surveys and investigations, including exploration of foundation conditions, must be made and reported on. The necessary lands and riparian rights should be secured, in so far as possible, in advance of construction. Property damages and interferences with existing roads, bridges, railroad rights of way, etc., must be considered. All these matters take time and patience, so much in fact that occasionally the promoters abandon the project temporarily in order to allow time to assist in working out their problems.

Second.—Another handicap to water-power development is the general belief that such construction involves greater hazards and uncertainties as to final cost and amount of power actually available than the steam plant. There is doubtless good reason for this belief. Severe floods may occur at inopportune times during the construction period; the exact cost of riparian rights and property damages can never be predicted in advance; and water-power plants are subject to a variation in annual output, due, of course, to the variation from year to year in the amount of rainfall and run-off.

Third.—The actual construction of a complete hydro-electric plant involving a dam, intake, conduits, power house, transmission lines and sub-stations

* Cons. Engr., Washington, D. C.

usually requires a longer time than the erection of a steam plant of equal capacity.

Fourth.—The initial or capital cost of water-power development is inherently greater than that of steam-power plants. A larger amount of capital therefore must be raised.

Fifth.—The complete regulation of a stream is necessary before water power becomes independent of steam reserves.

Sixth.—There is a wide difference of opinion between the promoter and the banker on the one hand, and the public utility commissions on the other, as to a fair compensation for the effort required to promote and develop water power.

Recognizing these and other similar conditions as actual handicaps to water-power development, what then are the chief economic reasons for pursuing the construction of hydro-electric plants in districts where steam-power systems are already in control of the market?

In answer to this question there are perhaps only two important reasons. In the first place, water-power developments having ample storage reservoirs are peculiarly adapted to operation under low-load factors, whereas steam plants are most economical when carrying full loads continuously. As an explanation of this, it is known that when steam turbines are operated during peak-load periods only, heavy standby losses are incurred during the remainder of the day as boilers of corresponding capacity must be banked and held ready for service. On the other hand, hydro-electric units, when idle, are free from practically all standby charges except fixed charges; and in the matter of fixed charges it may be said that the capital cost per kilowatt of such additional capacity in a water-power plant such as would be required for peak-load service only, is usually about one-half the cost of peak-load capacity in a steam plant.

The other important incentive to hydro-electric development is the desire to pre-empt power sites against the time when the cost of fuels will be higher and the needs for power service much greater than they are to-day. The appropriation of water-power sites by steam-electric systems also tends to shut off the possibility of destructive competition.

The general demand for power in this country has been doubling every five or six years, and there is no present indication that this rate of increase will be lessened in the future; hence the necessity for a more complete utilization of water-power resources. Ten years ago hydro-electric securities were acceptable to only a few investors and only a limited number of bankers would take any interest whatever in the promotion of such projects. As water-power development was considered hazardous, it was undertaken only by such companies as could be financed without appeal to the investing public. This limited the activity in hydro-electric construction to a comparatively small number of corporations, and perhaps indirectly stimulated the popular belief that the country's water-power resources, like its forests, should be withheld for a time from development—the reason for this belief being that otherwise all water powers would ultimately come under the control of the few corporations interested. This kind of conservation, however, is now obsolete. There is no longer any desire, or necessity for conserving water power by prohibiting its

development, but instead it is the limited supply of coal, gas, and oil that should be conserved through the utilization of water power. What then can be done to promote and encourage hydro-electric development?

It is true that a few of the handicaps to water-power development are inherent and cannot be corrected. They can be offset, however, but this will require adequate legislation and perhaps slight changes in the policies now commonly in vogue among public service commissions throughout the country. The speaker hesitates to open this phase of the subject because doubtless many believe that there is already too much legislation involving the control of power development. Granting, however, that some sort of legislative control is the wish of the majority of the people, the following observations from personal experience are made for the consideration of those who are interested especially in this subject.

In the first place the Federal Power Law enacted in 1920 has done much to create beneficial interest in National water-power resources; it has facilitated power development on public lands and on navigable rivers, at the same time adequately protecting the interests of the general public. However, from the viewpoint of many who are willing to engage in water-power development, the law appears either unsatisfactory or inadequate in its definition of those streams which are to be subject to Federal control, for it handicaps by taxes and regulations, even though moderate, many new projects, in comparison with those older developments which were constructed before 1920 and which are therefore exempt from such taxes and regulations. Parts of the present law permit honest differences of opinion as to the actual intent of the law-makers. Consequently, differences of opinion exist with regard to the regulations now in force. This condition of uncertainty must be corrected either by amendment to the law or by precedent in its administration.

Another way by which water-power development can be encouraged is by correcting the present inadequate State legislation, especially with respect to rights of eminent domain and the methods of procedure under the rights conveyed by law to water-power companies. Here again precedent is lacking, and honest differences of legal opinion are possible, so that expensive and unjustifiable delays often result, even though the intent of the law is clear. In one particular instance where the dam proposed by a power company was going to flood out the pumping plant of a water-supply company, it was necessary for the power company to buy and undertake to operate the entire plant and distributing system of the water company merely to avoid the injunction and delay that would have resulted if condemnation proceedings had been instituted to secure permission to reconstruct the pumping plant at a new site.

In another case where it was necessary for a power company to reconstruct a highway bridge at a higher elevation, it was found that the County Commissioners owned the structure; the County Court took jurisdiction as to the relocation of the bridge and highways; the State Highway Commission had jurisdiction over the changes in approaches to the bridge, as it was on a road designated as a State Highway; the State Water Supply Commission also had authority over all construction operations in or across streams; and the United States War Department likewise took jurisdiction, as the river was

considered a navigable stream because at one time it had been used to float logs and rafts. Fortunately, while efforts were being made without success to bring about some agreement in the requirements of all these authorities, a new State law was passed setting up a commission which would have final power in such matters in so far as the State was concerned; and thereafter the necessary permission was promptly secured to raise the bridge. This, however, did not prevent all the other authorities from holding hearings, nor relieve the power company from attending such hearings and presenting evidence, although the procedure in each case was perhaps more or less perfunctory.

With respect to the taking of private property by condemnation procedure, the laws of some States permit the filing of a bond with the Court to cover all possible damages, and thereafter allow the immediate occupancy of the property in question. This is satisfactory. In other States, however, the petition to the Court must be advertised, subsequent appraisal made by jury, damages assessed by the Court, and payment actually made by the power company before the property can be occupied. As appeals are possible to superior Courts, the process is one which is subject to long delay, and the law is therefore of no real benefit to the power company, which must have the property in order to complete its construction work. Moreover, a large number of such cases filed at one time with the ordinary County Court would completely clog the docket with the result that prompt action could not possibly be secured.

Another law conveying the right of eminent domain limits the cases which can be heard to those involving damages of more than $3 000. This gives the power company no recourse except to pay a recalcitrant land owner at least this amount for his property irrespective of its size or value. Some States are now enacting water-power laws patterned after the Federal Power Law; these can be made to embody provisions which will materially assist a water-power company in determining more rapidly and fairly the amount of its liability for property damage. Care should be taken, however, to avoid undue duplication either of authority, or of taxation, or of control over water-power development.

A third opportunity to favor water-power construction rests with the public service commissions. These commissions have done much in the past to stabilize public utilities. One of their general policies is to prevent destructive competition between utilities by allotting each particular district by charter or franchise to only one power company, and then to require as good service as is practicable, allowing rates which will permit a fair return on invested capital. This policy is constructive and usually entirely satisfactory. It does not, however, encourage water-power development in that no premium is put on the economy that might result from lower generating costs. There is no intention to imply that the average power company is not interested in the efficiency of its steam turbines or in the cost of generating power, but the difference in cost of the cheapest water power as compared with the most efficiently generated steam power is so small a proportion of the retail price to the consumer that there is no real incentive urging the company to make the necessary effort to develop water power. If the desire to pre-empt water-power sites and to utilize

hydro-electric power for peak-load service is not alone sufficient to bring about the development of all the better water powers, then, with a view to saving coal, it may some day be necessary for the public service commissions to insist that an electric utility occupying territory in which water-power possibilities exist shall develop these powers for the purpose of supplying their normal increases in demand rather than make further additions to their existing steam plants, allowing adequate rates, of course, to compensate the company.

Such a policy, practically developed and administered, and supplemented by modifications in Federal and State laws, particularly with a view to facilitating the exercise of rights of eminent domain, would result in the prompt utilization of the nation's water-power resources and thereby the conservation of its fuels.

W. S. Lee,* M. Am. Soc. C. E.—The economics of hydro-electric development is very prominently before engineers of this country. Probably the next ten to twenty years will be the hydro-electric decades in America. People are thinking more about hydro-electric development, more about economics and conservation.

This paper, although covering a great many subjects, in harmony with its broad scope could not be devoted to a detailed discussion of these various topics.

There are three principal items to be considered in such a project: First, the site; second, the money; and third, the market. If one is missing the project will not be successful, or perhaps may not be constructed.

The variations of stream flow that prevail in Michigan, prevail in the Southern Appalachian slope to a much greater extent; there are variations of 200 000 sec-ft. for a drainage area of 1 400 sq. miles. It can readily be seen that to provide for this divergency is difficult and expensive. Hydro-electric development requires a co-ordination of the companies to smooth out these irregularities, an interconnection to assist one another in the event of a depression or a peak in their power supply. This does not necessarily mean that these companies must be one; there can be several companies, but there must be some plans of interconnection between them, in order for each to obtain the greatest amount of energy or the best value out of its plant. The Southeastern section has believed in interconnection for years. The speaker urged it in 1910 before the American Institute of Electrical Engineers. Years ago it was found that these variations of stream flow in one section could be used to help out where the discharge was more uniform. The author's treatment of this phase of the subject is impressive.

He has discussed the use of water. When the water in the tail-race is turbulent, no matter who the designer was, nor what the type of hydraulic apparatus used, energy is being wasted. When the discharge flows in a quiescent state, with a reasonable velocity, an economical point in the design of the tail-race has been reached.

In the design of dams it is important to take care of the enormous overflow and, above all else, to prevent sliding, especially if large quantities of water must overflow and pound the structure.

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The speaker is opposed to more legislation and does not see how the public can be hurt by the public utilities, such as power companies, if the rate-making power is left to the commission. Engineers who are interested in developing cheap power from oil and coal are standing on guard saying, "You cannot put rates too high, or we will compete, using steam". There are many good laws on the statute books; among them is the Federal Water Power Act. This is especially important as about 75% of the undeveloped power of the United States is on Government property; but it is not a good law if it fails to cover the property that the United States owns or holds, and if the Government can turn every spring or brook into a navigable stream by calling it such. The Government owns Muscle Shoals *in toto*—not water rights—and it is proposed to give it to an individual for an individual use; on the other hand, if an individual wants to build a plant on some stream that floated a log back in 1850 he must go to the Federal Government for permission, and turn the plant back in 50 years, after furnishing all the money for development. Such laws shake one's faith in all this additional legislation.

Building a hydro-electric plant is not like building a steel office building, or a structure where there is no flood during construction. A well-trained organization can effect a saving of 25% in developing a hydro-electric plant by co-ordinating the design, construction, and erection. There are many ways to do this and not only effect immense savings, but insure the safety of apparatus from flood while building and shorten the time of completion. The construction plans and erection plan should be arranged so as to escape the danger of the flood waters as quickly as possible, and thus save money.

Brent S. Drane,* M. Am. Soc. C. E.—Just two thoughts occur to the speaker; they are hardly technical but apply rather to the engineer's duty to himself and to the public as a citizen. Professor Mead has emphasized the dependence of the profession on some continuous agency for furnishing stream-flow data; and that must be, as all know, a governmental contribution to the professional need. He has mentioned the fact that the Oklahoma record, which would have been so valuable in the case he cited, was terminated in 1908, just before extremely significant data might have been secured. It is a costly coincidence that in 1908 a spirit of economy in the Federal Administration caused the abolition of the Water Resources Branch of the United States Geological Survey. The cost of this economy to the people of Oklahoma City has been vividly illustrated by Professor Mead.

The speaker is convinced that the public grows in intelligence year by year, and responds more and more to the thought of the engineer. The profession is under obligation to this public constantly to impress on it that it owes, through governmental agency, a fundamental duty in providing basic data for the use of technical experts who serve the public, thereby effecting public economy.

The other matter of importance was mentioned by Mr. Hawley in his admirable discussion. A majority of engineers who deal with hydro-electric

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projects at all deal with relatively small independent plants. Do they not, as a profession, owe it to the public, to promote the conception of a power stream as a unit—the conception of the stream ideally developed as having a certain potential economic value which may be considered as a definite amount of capital wealth, and the attendant realization that anything less than a development of this kind means the destruction of capital wealth? Every one is familiar with the disastrous effect, when a relatively uneconomic development interferes with that of one or more preferable sites and with the complete attainment of the stream's potential value. Evidently, the ideal attitude may involve sacrifice on the part of the individual engineer; but is it not true that the members of the profession as good citizens do owe just this duty to the public as part of their contribution to the conservation of National wealth?

Daniel W. Mead,* M. Am. Soc. C. E. (by letter).—In treating this broad subject, the attempt was made to condense the paper so that it might afford a general but complete outline of the entire question. Being general in character, the outline is subject to modifications when applied to any particular problem, for in such cases certain factors may become insignificant, while others may be accentuated.

The writer limited his analysis in a measure to independent hydro-electric developments, although most factors in the outline will apply equally well to a development which is to be a part of a larger power system. As part of a system, the proposed development must necessarily be considered in its relation to the system as a whole, and the aspects of the problem may be considerably changed. Under such conditions the greatest change is perhaps the reduction in the uncertainties as regards the market, the cost of financing, the development of business, the cost of management and operation, and the proportion of possible output that can be utilized.

When a business of considerable size is once developed, its extension is accomplished with much less uncertainty than attends the establishment of a new business. Additions to a profitable business can be financed at much less expense than the development of a new business that is entirely or largely speculative. The cost of management and operation can be based on actual local experience, and the nature of the load and its probable trend of development will be based on conditions fairly well established; thus, the real value of the new plant to the business is more readily and exactly determined and there is less hazard in many phases of the problem. As a part of a system a hydro-electric plant may be developed to a greater machine capacity than would be advisable for a similar independent plant, for it may prove economical, as stated in the discussion, to utilize such a plant for the base load during high water and, if pondage is available, reserve it for carrying peak loads during low water, as under such circumstances capacity can be secured more cheaply than by the installation of additional steam-electric capacity. When a hydro-electric plant is a part of a system, other factors besides the load may be subject to similar modification.

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The writer has read with interest the discussions of the subject and recognizes that the various illustrations used in the paper (including Tables 2 and 3 and Figs. 5 and 6, also, Appendix D) are special cases which can be applied only in principle to similar studies. In general, the discussions are elaborations of various phases of the question which might be indefinitely extended, as the subject if treated in detail would require several volumes.

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Paper No. 1556

INCREASING THE CAPACITY OF EXISTING STREETS*

By Arthur S. Tuttle,† M. Am. Soc. C. E.

With Discussion by Messrs. Harvey W. Corbett, C. D. Hill and Jacob L. Crane, Jr., Harland Bartholomew, T. Kennard Thomson, W. W. Crosby, George S. Davison, George T. Seabury, E. P. Goodrich, John A. Miller, Jr., William T. Lyle, A. C. Janni, C. E. Grunsky, and Arthur S. Tuttle.

It is only a few years since the public depended wholly on the horse-drawn vehicle for free-moving transportation. The introduction and popularization of the motor car has created a problem which the authorities in nearly every city are endeavoring to solve in order that street capacity may be made adequate for the ever-increasing demands of traffic.

City planning in New York began in 1811, when a commission appointed by Mayor DeWitt Clinton prepared a map for that part of the old city—now the Borough of Manhattan—extending from the developed area south of Houston Street and Greenwich Village northerly to West 155th Street, a distance of about 7 miles. This map made provision for a series of parallel north and south avenues, each 100 ft. wide, spaced 610 to 920 ft. apart, and at right angles a series of parallel east and west streets, 200 ft. apart, with widths of 60 ft., except at intervals of seven to fifteen blocks where widths of 100 ft. were established. This plan, later extended northward to the Harlem River, largely constitutes the Manhattan Plan of to-day. Until recently, it was considered to have provided more than an abundance of street space. As a result, invasions have been permitted to such an extent as seriously to impair the usefulness of a plan which, in breadth of vision, compares favorably with present-day planning in any of the large cities. It would seem that, although the city inherited a layout of great merit, the inheritance was not valued sufficiently by the citizens of later years, otherwise, it would be much easier to remove the various impediments to traffic that have been introduced.

* Presented at the meeting of the City Planning Division, of the Society, January 17, 1924.

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Present conditions in New York are probably typical of those of other large cities, but more acute. Statistics on the traffic across the East River bridges indicate an average annual increase of about 20% for the last 10 years, and the Police Commissioner states that the number of vehicles in the city is increasing at the rate of about 30% annually. From a recent study of all available data, it has been estimated by the writer that 15% represents a conservative estimate of the present annual increase in traffic in the congested districts, with the probability that the rate would be much higher if the street capacity was more adequate.

Police regulations have been carried as far as seems practicable under present conditions. One-way east and west traffic is in force through the congested districts and the two main north and south arteries are controlled by signal systems. It is acknowledged that further relief is urgently needed in order to cope with the present, to say nothing of the constantly increasing, traffic needs.

The widening of streets after they have been fully improved, or the breaking through of new avenues heretofore observed is expensive, usually involves the complete destruction of the buildings affected as well as heavy consequential damages to land, and frequently leaves the frontage so badly gored as to be unsuitable for further substantial use.

It has been estimated that to cut a new express street 100 ft. wide through the heart of the Borough of Manhattan would cost about $25 000 000 per mile. Excess condemnation, for the purpose of salvaging remnants, would temporarily add still greater burdens and would involve so much capital that such a project in this part of the city would be largely impracticable.

In order to provide the relief now demanded without embarking on a plan involving the utilization of unduly large areas devoted to private use and without destroying valuable buildings, a number of expedients have been, or are being, considered, as follows:

1.—Requiring new buildings to be set back to lines proposed as the ultimate limits of the widened street.
2.—Widening of roadways.
3.—More drastic traffic regulations.
4.—Removal of roadway encroachments.
5.—By-passing of traffic.
6.—Introduction of arcades.

1.—Set-Back Ordinance.—In some cities, street widenings have been accomplished through legislation prohibiting the erection of a building other than in conformity with the lines for a new street or for the widening of an existing street, thereby limiting the cost of widening a street which had been fully built up to the value of the land, provided that the proceeding was deferred until after all the old buildings had been replaced. In the meantime, it seems probable that the land needed for street purposes would have become burdened with public easements to such an extent as to have only a nominal value. Such a practice was observed in New York until 1893, when its legality was contested in the Courts. The opinion rendered in this case was as follows:

"Whenever a law deprives the owner of the beneficial use and free enjoyment of his property, or imposes restraints upon such use and enjoyment that materially affects its value, without legal process or compensation, it deprives him of his property within the meaning of the Constitution, and, although the police and other powers of government may sometimes incidentally affect property rights, these powers can only be exercised to promote the public good, and are always subject to judicial scrutiny.

"The provision of the New York Consolidation Act (Section 677, Chapter 410, Laws of 1882), which declares that no compensation shall be allowed to the owner of land taken for a street for any building erected or placed thereon after the filing of a map of the street as prescribed by the Act (Section 672) by its terms, imposes a restriction upon the use of the land, which amounts to an encumbrance, and so is unconstitutional."

The unconstitutionality of such a law seems clear both on the basis of the Federal as well as of the State Constitutions, and the decision has since been accepted in New York. Efforts elsewhere to break down such a law will be made when the land and building values are sufficiently high to warrant litigation and must inevitably meet with success.

In this connection, it might be noted that, under the Zoning Act, the city authorities of New York have recently imposed a 15-ft. set-back on the highest class of residential property and now have under consideration the establishment of a 10-ft. set-back for residential areas where the restrictive conditions are less severe. These restrictions are being enforced under the police powers of the State and are designed to protect a particular type of development to which an area seems best adapted. It does not seem probable that any such requirement could be successfully imposed where development had been well advanced and was primarily of another type, or that similar means of relief could apply to congested sections.

2.—Widening of Roadways.—During 1908, the congestion of traffic on Fifth Avenue, which is the only north and south street in Lower Manhattan free from obstruction, became so acute that the roadway between 25th Street and 47th Street was widened from 40 ft. to 55 ft., the sidewalk widths being decreased from 30 ft. to 22.5 ft. At that time, Fifth Avenue was a residential street. This improvement involved the removal of entrances, steps, and fences. The reconstruction which at once took place converted it into a high-class business thoroughfare and was followed by an extension of the roadway widening to include the entire length of the street south of Central Park. This widening marked the first important move toward the freeing of the streets as far as possible to meet the needs of vehicular traffic without increasing the street width.

A large number of similar improvements in the more congested parts of the Borough of Manhattan have since been made, particularly in streets having a width of 60 ft., with 30-ft. roadways and 15-ft. sidewalks, originally developed for residential use. A substantial part of the sidewalk area has been appropriated in a manner similar to that already described in Fifth Avenue. The width of these roadways is being increased to 34 ft. with repaving improvements and, through the removal of encroachments, practically all the space available to pedestrians is being conserved. This has caused a marked relief in districts having light traffic, as it permits two lanes of vehicles

moving in opposite directions and a lane of traffic parked at each curb. In districts where the streets are used by large trucks, it is evident that traffic must still be seriously impeded.

The expediency of still further decreasing the sidewalk allowance, which now generally averages from 13 ft. to 20 ft. in width, has recently been questioned, but it is evident that the resulting gain would not accomplish much relief.

In the planning of vehicular tunnels where construction costs are high, careful consideration has been given to the kind of traffic and the space required for its accommodation. It does not appear that heretofore any such serious consideration has been given to the planning of a street, but by reason of the great expense involved in the congested areas, it seems evident that this phase of the problem should be considered. The number of lanes of traffic must be determined, and space must be provided for each.

Traffic regulation requirements seem to call for a permanent demarcation between lanes used for movement in opposite directions, and it may be expected that eventually each lane will be clearly marked on the roadways. This being true, it follows that street widths should be fixed in units which will accommodate two lines of parked traffic and an even number of moving lines, and should not be increased at random in small units as heretofore has generally been done.

3.—Traffic Regulation.—The introduction of traffic regulation at 42d Street and Fifth Avenue during 1903 has been followed not only by its extension to all busy intersections throughout the city, but also by imposing one-way traffic rules on the narrower streets where there is congestion and by establishing traffic-tower control over some of the most important arteries. These methods of handling the traffic problem have been improved to a point of efficiency which is believed to be unsurpassed. The Police Commissioner has recently announced, however, that regulation under present conditions has reached its limit and that relief must come through the introduction of new devices.

Pending the provision for more street area, it would seem that a substantial gain in capacity might be obtained through the limitation of arterial streets to one-way traffic and the introduction of the block system of control, the latter being dependent on the former. One-way traffic regulation requires a re-routing of surface cars and bus lines so that all movement will flow in one direction; it can only be effected through co-operation between the municipal authorities and the operating companies. Its adoption would doubtless meet with opposition from the business interests affected which, however, are bound to suffer through the perpetuation of present conditions, to say nothing of the certainty of continuously increasing congestion unless relief is found. The block system of operation resembles that used for railroads and, if properly controlled, should permit traffic in the controlled area to proceed continuously through it without halting, the section thus controlled becoming a semi-expressway. This arrangement, with due allowance for increasing the distance between cars as required for safety, should have the effect of speeding up the movement at least 25%, and thus afford a substantial increase in street capacity.

4.—Removal of Roadway Encroachments.—Until recently, roadways have been regarded as having surplus width and their invasion, beginning with surface car tracks and steam railroads, has extended to elevated railroad columns, subway station entrances, subway ventilator gratings, and malls. These utilities have served in the growth of the city and when the municipal authorities announced about four years ago that the day for further extension of elevated railroads in the city had passed, the statement was at first not taken seriously. However, not only is this policy now generally accepted, but active efforts are being made to restore the streets to the use for which they were intended.

The seriousness of the situation is strikingly illustrated by recent traffic counts of the north and south movement across 42d Street at all the arteries between the Hudson and East Rivers. From these studies it was found that the movement ranges from a headway of about 5 sec. in unobstructed Fifth Avenue to about 25 sec. in streets having major obstructions, the average headway for all being about 9 sec. It would appear, therefore, that if all the obstructions, such as surface tracks, elevated railroad columns, and the West Side Division of the New York Central, which is operated at grade, were removed, the capacity of the north and south streets in the most congested part of Manhattan could be increased about 80 per cent.

With this object in view, the President of the Borough of Manhattan has recently proposed the removal of the Sixth Avenue Elevated Railroad and the substitution of a subway, thereby freeing the street, except for surface cars, to vehicular use and restoring the light and air of which it is now to some extent deprived. (Fig. 1.) The adjacent property owners have shown a keen appreciation of the benefits that would follow such an improvement, even to the extent of expressing a willingness to bear the cost of the improvement in the belief that the benefits would more than equal the expense.

The surface tracks of the New York Central Railroad have constituted a serious problem, the solution of which cannot long be deferred in the interest of either the public, whose safety and convenience are seriously prejudiced, or the corporation, which is subjected to great difficulties in operation.

The economic advantages of the surface car over the bus and taxicab in street capacity used, have been to a considerable extent lost in the crowded sections of the city by reason of the slowing down of speed as the result of the absence of freedom of movements, which slowing down is also shared with all the traffic using the same street.

The removal of subway malls and station entrances from the middle of the roadway along the line of a subway opened for operation about ten years ago (Fig. 2) is now seriously proposed and unquestionably will be effected in the near future.

The City of New York has learned its lesson, and although it has reaped decided advantages by reason of the treatment which has been permitted with respect to its roadways, its later experience should serve as a warning to other municipalities with similar problems, the correction of which is certain to be both costly and slow.

FIG. 1.—VIEW OF SIXTH AVENUE, NEW YORK, N. Y., LOOKING NORTH BETWEEN 35TH AND 36TH STREETS.

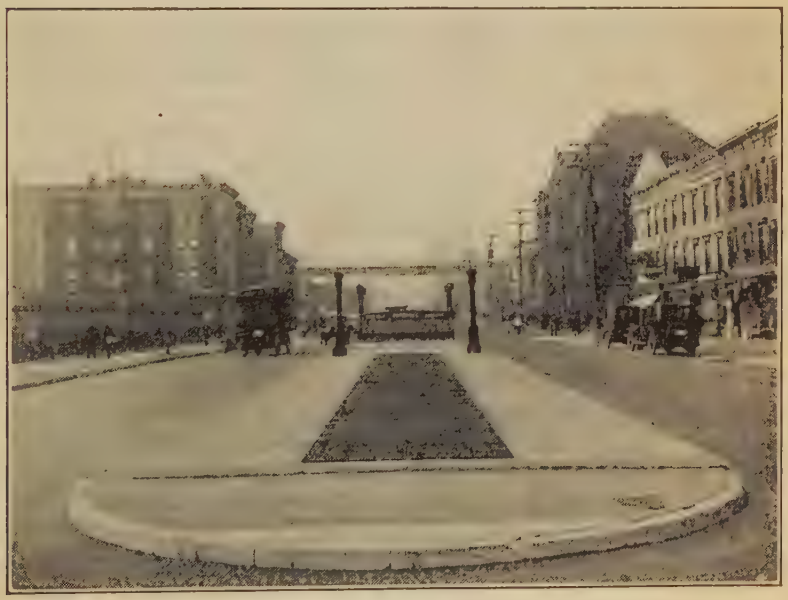

FIG. 2.—VIEW AT FOURTH AVENUE AND 36TH STREET, BROOKLYN, N. Y.

5.—By-Passing of Traffic.—One of the most promising fields for relief lies in the diversion of through traffic from congested sections into streets giving greater freedom of movement or more economical increase of highway capacity. This remedy is being applied in New York as far as possible in the planning of boulevards in the outlying districts, but is only available to a limited extent, as the greater part of the traffic that enters the city is seeking the areas of congestion.

A project for an expressway along the Hudson River water-front has been suggested from time to time for many years and has recently been revived by the writer in a more comprehensive way. Traffic on Riverside Drive south of its terminus at West 72d Street is dispersed through various arteries, none of which stands out pre-eminently. Between 72d Street and 59th Street, the water-front is occupied by the 60th Street Yard of the New York Central Railroad Company. Between 59th Street and the Battery, except for a short distance south of 42d Street, the city has acquired a strip of land about 150 ft. wide for a marginal way to serve the water-front. This strip in turn is flanked by West Street, with a width of about 100 ft., designed for the accommodation of the vast trucking business incidental to the service of the piers. The marginal way is mostly paved, and this, as well as West Street, is usually congested with a tangled mass of vehicles awaiting an opportunity to discharge or receive the pier freight. It is proposed to construct a viaduct over the 60th Street railroad yard and a shed over the marginal way to a depth of about 75 ft. Over this it is proposed to construct an expressway 100 ft. wide with a 25-ft. overhang toward West Street. This arrangement would make it possible to distribute pier freight from a landing platform extending practically the entire length of West Street, against which, and in the covered area outside the street, the trucks could back up for loading or unloading, thereby avoiding the present confusion, clearing West Street, and conserving time. It would seem that such a project would greatly decrease the present cost of handling freight at the water-front, and, incidentally, make it possible to recoup a considerable part of the cost of the express street. Such a street would be provided with ramps to grade at intervals of about a mile. With its ten moving traffic lanes, it would provide a capacity more than twice as great as that of Fifth Avenue; assuming a conservative increase in speed, it would seem that the new street would accommodate three times as many vehicles as Fifth Avenue.

In the development of such a project, it would also seem feasible to move the express street up to a third level and to introduce in the middle level a marginal water-front railroad which could easily be connected with warehouses on the easterly side of West Street and provided with facilities for directly receiving or discharging marine or local freight (Fig. 3).

6.—Arcaded Sidewalks.—In order to provide more space, it would be practicable to double-deck streets, but the introduction of such elevated structures, except along the water-front, is not regarded by the writer with favor, his objections being similar to those against elevated railroads. Furthermore, double-decking would depreciate property values on the lower level even

more than elevated structures, and the construction would involve costly building and consequential damage in ramping to grade.

There lies an intermediate procedure between this course and an actual street widening, that is, to provide a sidewalk in an arcade back of the street line and wholly within the fronting buildings.

The only known example of such an arrangement in the United States for the purpose of devoting to roadway use what would ordinarily be a sidewalk, is noted* by the late Nelson P. Lewis, M. Am. Soc. C. E., as follows:

"Philadelphia again furnishes a notable example. Fifteenth Street, between Market Street and South Penn Square, has a width of 50 ft., with a roadway of 26 ft. and sidewalks of 12 ft. each. The curb on the east side was set back 11 ft., or within 1 ft. of the street line, while a sidewalk 19 ft. in width was provided back of the new curb line extending 18 ft. under the buildings. This improvement was also extended eastward along the north side of South Penn Square between 15th and Broad Streets. The arcade thus formed has been treated uniformly, the supporting piers are regularly spaced and all of the same size, and the results of this treatment have been very satisfactory, the shops fronting on the arcade appearing to be desirable and probably commanding good rentals. The total length of this arcade on both streets is 335 ft., and the cost to the City in damages paid for the easements and the reduction of

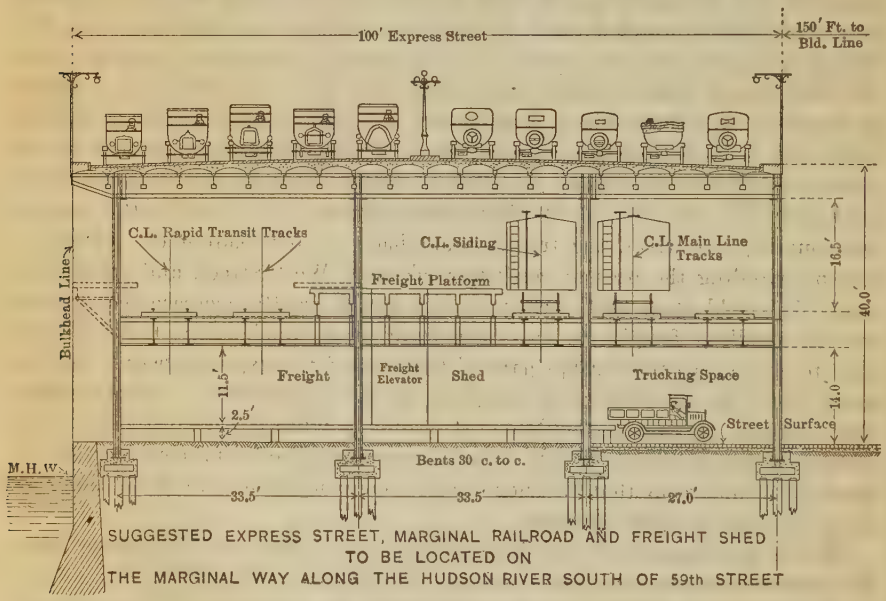

FIG. 3.

available floor space in the buildings amounted to $193 000. Consideration is now being given to the extension of this arcade along the east side of 15th Street to Chestnut Street, a distance of about 200 ft. It should be pointed out, however, that such treatment cannot be successful unless the arcades are of uniform height and unless the supporting columns and the façades of the buildings themselves are harmoniously designed."

* Paper presented at the National Conference on City Planning, 1917.

Fig. 4.—View of Municipal Building, New York, N. Y., Showing Arcade Between Park Row and Center Street.

Fig. 5.—View of Proposed Arcade on Vesey Street, New York, N. Y.

In order to secure the desired floor space, the Municipal Building in New York was arched over Chambers Street, through the block between Park Row and Center Street, the sidewalks being placed in an arcade (Fig. 4). In this instance, however, there was no occasion for equipping the arcade with show windows such as would ordinarily be required, and there was no damage to abutting property. A somewhat similar treatment is proposed for the building to be erected by the New York Central Railroad Company in Park Avenue, between 45th Street and 46th Street, as a part of the extensive improvement involving a high-level street along the easterly side of the Grand Central Terminal.

Possibly, the practical advantages of the arcade are better exemplified by the building now being erected by the New York Telephone Company, extending along the northerly side of Vesey Street, from West Street to Washington Street. Vesey Street here has a width of about 56 ft., and it was proposed by the city authorities to take advantage of the opportunity which would be afforded by the removal of the old buildings to increase the street width to about 80 ft. The Telephone Company considered the project to be objectionable on account of the restricted space which would result for building purposes and, as a compromise, proposed that if the street widening was abandoned, it would give the city sidewalk privileges within the building to a width of about 20 ft., as shown in Fig. 5. Hitherto, the street arcade had been regarded as somewhat impracticable because of what appeared to be a serious difficulty in acquiring, under formal proceedings, a form of title that would enable the city authorities to construct a sidewalk without at the same time depriving the owner of rights that might be essential to a convenient or proper use of his property. The studies made in this instance disclosed the practicability of effecting an adjustment fair to both parties, and of establishing the legality of the procedure. It should be noted, however, that any such proceeding conducted on a scale which affected several properties would have to be developed along lines adapted to each individual parcel.

In Southern Europe, the arcaded sidewalk seems to be quite common, particularly in cities where the climatic conditions are such as to make its use advantageous as a protection from severe snowstorms. The most notable example is the rue de Rivoli in Paris; this street has a width of about 70 ft. and the buildings on the northerly side, through a considerable part of its length, have been arcaded and the roadway correspondingly widened. Other excellent examples in Paris are found in the rue Castiglione and rue des Pyramides. The former street has a width of about 75 ft. and the latter of about 40 ft., and the sidewalks of both are located wholly back of the building lines.

It seems evident from these examples, as in Philadelphia, that the arcade has not detracted from the business advantages of the property, and it seems probable that the value of business property has increased. The type of skeleton frame building now in common use for office and commercial purposes lends itself admirably to arcade development, and with an effort to adapt the arcade plan to each, it would seem that ordinarily its introduction should not involve a large cost for the rights required.

The automobile parking problem is also as vexatious as that relating to moving traffic. The line of parked cars along each curb, whether scattered or clustered, whether standing for a few minutes or for hours, constitutes to all intents and purposes the occupancy of what might otherwise be two lanes for moving traffic. If the arcade principle comes to be accepted as the most practicable remedy for congestion where property values are high, it would seem logical to extend it in such a way as to place the sidewalk on the second level, with bridges across intersected streets, and to devote the lower arcade level to parking use, thus not only increasing the roadway space, but also separating the grades for pedestrian and vehicular traffic, thus greatly increasing the safety of walkers and speeding up vehicles.

In a recent study of the advantages of the arcade treatment for the north and south avenues of New York, the conclusion was reached that the assignment of sidewalk space to roadway use would nearly double the number of available traffic lanes, and that from every point of view this method was the most practicable for the economical and comprehensive remedying of congestion.

Heretofore, the traffic relief problem has been regarded as one in which the Police Department was primarily concerned, but it would seem that it has now assumed such proportions that the engineering service must take the lead in providing a solution. The recognition of this responsibility by the profession should bring about the freeing of the streets for the traffic needs of to-day and the days to come.

DISCUSSION

HARVEY W. CORBETT,* ESQ.—Mr. Tuttle has presented an interesting paper, but he does not go far enough. He blames traffic on the automobile. Everybody blames the automobile. The automobile is like the weather Mark Twain spoke about, "Everybody talks about it, but nobody does anything to improve it." The automobile is not the only cause to which traffic conditions are attributable.

It is a common statement that high buildings are erected in New York because there is no room for expansion anywhere except upward; the lower end of Manhattan appears to bear out this impression. One can readily understand that the traffic congestion is not wholly a question of automobiles or people; it also concerns the bulk of the building in relation to the street. The street is the artery which feeds the building with people, goods, food, etc. If the bulk of the building passes beyond a certain relation to the street, congestion is bound to result. Each building erected is five, ten, or even twenty times the bulk of the one it replaces, yet nothing is done with the street. That condition must be met; whether it is right or wrong, it is a condition. What can be done to increase the capacity of the existing streets? The city carries three easily divisible kinds of traffic: Foot, wheel, and rail. If the traffic problem is to be solved, logical divisions between those three types of traffic which do not belong together must be made.

For many years, the grade crossing has been considered as a menace. It seems logical that rail, wheel, and foot traffic should be separated and placed in such relation to each other that they do not cross and conflict. As rail traffic is the heaviest in weight, it should go underground. It is already underground in the subways. The operation of surface car lines is becoming less and less successful as a business proposition, and will be abandoned in time. Wheel traffic is gradually absorbing what formerly was carried by rail, because more people are riding in motors, and less on street cars and in subways. It is also absorbing foot traffic; there are more people riding than walking. Wheel traffic is the one great, growing form of traffic. Even freight which in past years was hauled in trains, is being carried not only into the city, but after it arrives there, in wheel vehicles. The solution of the difficulty is, therefore, simple: Use the present surface of the city for wheel traffic only, and raise the pedestrian traffic one story above the wheel traffic, as it is the lightest and easiest to lift.

Fig. 6 shows the condition of an ordinary business street at peak load. Note the packages, bales, etc., with pedestrians trying to wiggle their way through, and a truck backed up to the curb cutting off three-fourths of the road. This can be seen on any of the cross streets at any time. A one-way street accommodates one line of traffic, as shown by the black car. It is necessary for trucks and vans to be parked, because that is part of the business of transportation of goods. It is an essential condition and cannot be changed by regulation or law.

* Archt. (Helmle & Corbett), New York, N. Y.

Fig. 7 illustrates the first step proposed, that of placing elevated sidewalks brought up from the grade at the avenues, thus allowing the trucks to park in the space formerly used for sidewalks. Such an arrangement would permit the movement of three lines of cars, which would increase the capacity 200 per cent.

Fig. 8 shows the third step, that of providing parking space beneath the building, thus increasing the traffic to six moving lines.

Fig. 9 shows the completed plan, the foot traffic remaining at the upper level, with the wheel traffic at the lower level. Building regulations should require all future buildings to be designed to provide for arcaded sidewalks within the building level.

Fig. 10 shows a city in its final aspect. All the people go to the upper level and walk through the city undisturbed by traffic conditions. The whole shopping zone would be available for them. The smaller parks would be raised to this same level, the space beneath being used as public parking space. In this arcaded city, the cross streets would pass under the avenues as shown. Automobiles crossing the city would take the lower level. By using only right-hand turns and under-cutting the avenues, it would be possible to reach any point of the city without being stopped, except by the slowing up due to traffic moving in the same direction.

It is difficult to approximate what the increase of traffic capacity would be. The speaker is satisfied that 2 000% increase of traffic over the present congested condition, is no exaggeration for such an improvement.

In New York and in all other cities where the traffic problem is acute, some fundamental principle should be established by which every proposed traffic solution should be tested.

C. D. HILL,* M. AM. SOC. C. E., AND JACOB L. CRANE, JR.,† M. AM. SOC. C. E.—It is desired to present herein information concerning two-level streets in Chicago, Ill. One two-level street, five blocks long, is in use; another, eight blocks long, is definitely projected; and, in addition, there are a number of unofficial proposals for the congested district. By checking the total capacity of one-level and two-level streets against their respective costs, the relative traffic-carrying efficiencies become apparent.

The principal elements of cost in these improvements, one-level as compared to two-level, are as follows:

	Land.	Construction.	Total.
Michigan Avenue, Two-Level.....	$5 000 000	$10 000 000	$15 000 000
Michigan Avenue, One-Level.....	5 000 000	4 000 000	9 000 000
South Water Street, Two-Level....	11 000 000	11 000 000	22 000 000
South Water Street, One-Level....	7 000 000	4 000 000	11 000 000

The nature of the foundation soil in Chicago necessitates elaborate and expensive systems of footings, extending as deep as 80 ft. below the grade of the lower level, to support the columns under the upper level. This accounts

* Chicago, Ill.
† Municipal Development Engr., Chicago, Ill.

FIG. 6.—VIEW OF ORDINARY BUSINESS STREET AT PEAK LOAD.

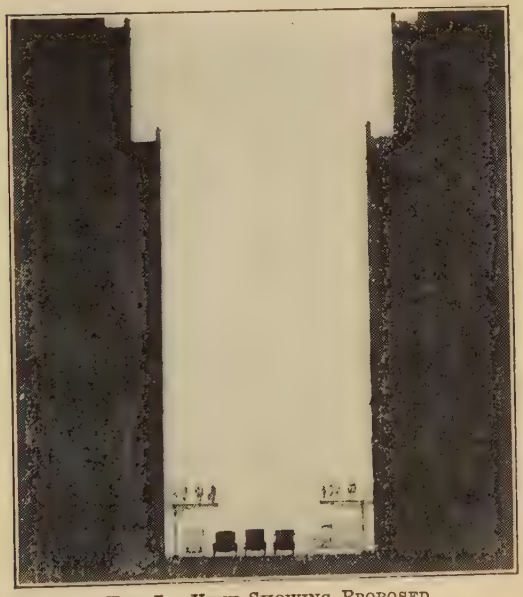

FIG. 7.—VIEW SHOWING PROPOSED ELEVATED SIDEWALKS.

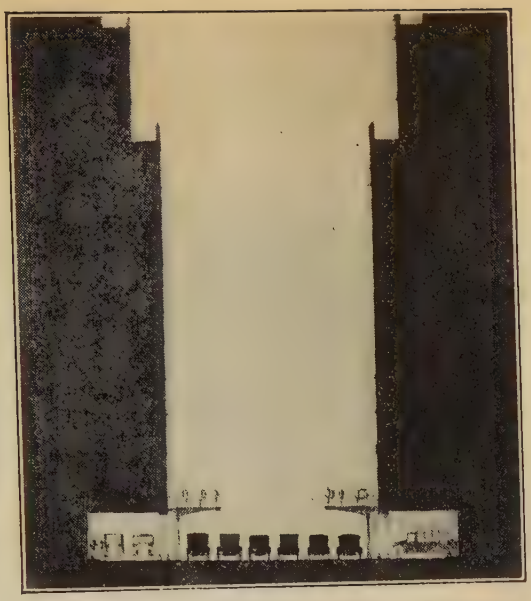

FIG. 8.—VIEW SHOWING PROVISION FOR PARKING SPACE BENEATH BUILDINGS.

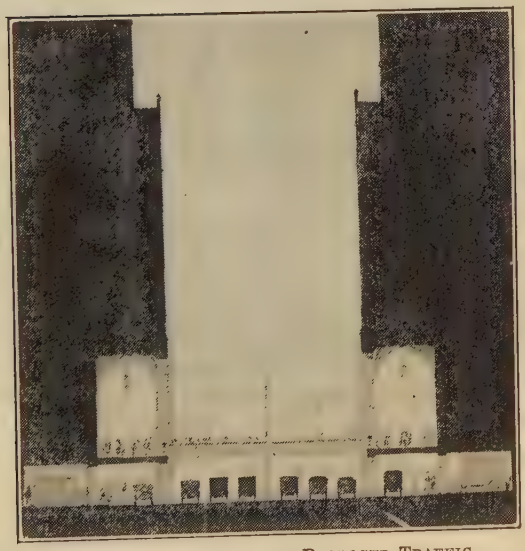

FIG. 9.—VIEW SHOWING PROPOSED TRAFFIC PLAN COMPLETED.

Fig. 10.—View of Proposed Arcaded City Streets.

for a large part of the extra construction cost in the existing and proposed two-level streets.

Michigan Avenue Two-Level Street.—The completed two-level street, known as the Michigan Avenue Improvement, or "Boulevard Link", was constructed to connect the upper end of Michigan Avenue south of the Chicago River directly through to the Lake Shore Drive, which, in turn, is a through boulevard to the northern end of the city and the North Shore suburbs. Formerly, the heavy traffic flowing from Michigan Avenue to Lake Shore Drive was compelled to take a narrow and devious by-pass over the Rush Street Bridge. The new connection (Figs. 11 and 12)* eliminates the by-pass and furnishes a wide, direct roadway, increasing many-fold the carrying capacity of this most important artery to and from the business center. Practically all the passenger traffic passes along the upper level, making it one of the greatest traffic carriers in the world. It accommodates more than 35 000 vehicles per day; within ten years, it will probably be carrying twice that number.

The carrying capacity of this upper level, however, would be much less if it were not for the lower-level arrangement, which removes the obstruction of a heavy traffic of teams and trucks along the line of Michigan Avenue and particularly across it. The largest single element in the cross-boulevard commercial traffic is that flowing to and from the freight yards of the Illinois Central and New York Central Railroads. From 4 000 to 5 000 tons of L. C. L. freight are hauled in and out of these yards each day, and the resulting traffic amounts to 5 000 or 6 000 teams, trucks, and automobiles. This traffic does not continue through the evening period of peak density for the upper-level traffic; still, if it passed across the upper level, it would reduce by one-third the capacity of that roadway. The lower-level traffic at the remaining cross-streets—Grand Avenue, Illinois, Austin, and North Water—totals another 5 000 vehicles, most of which are slow-moving. There is also a street-car line on Grand Avenue, and a railroad switching line on North Water Street. If this additional lower-level traffic was allowed to flow across and along the upper level, it would still further reduce its capacity.

The main purpose, therefore, of this two-level Michigan Avenue Improvement is the separation of the fast passenger vehicle traffic on the Boulevard from the slow commercial traffic which now moves along and across the lower level. In order to measure the money value of the time saved to the upper-level traffic by this separation, the following calculation has been made: To allow the lower-level traffic to flow across Michigan Avenue at the upper level, would make it necessary to interrupt the upper-level flow on the schedule of probably 1 min. closed to 2 min. open. If the traffic flow over this length were synchronized by one set of control signals, each vehicle would experience an average of one and one-half interruptions of an average duration of ½ min. each, during the working day, from 7:00 A. M. to 5:00 P. M. Taking this 10-hour upper-level flow at 36 000 vehicles (a figure which is probably already exceeded), there is an aggregate loss of time amounting to 27 000 vehicle-minutes, or 450 vehicle-hours, per day. At the same time, a still greater loss

* Furnished by the courtesy of the Chicago Plan Commission.

would be caused by the slowing down of all machines passing over this length. The average speed now attained with the upper level free from cross traffic is about 25 miles per hour, compared with an average of about 15 miles per hour that would be possible if this were a one-level improvement. This factor accounts for an average saving of 1 min., and gives an aggregate saving of 36 000 vehicle-minutes, or of 600 vehicle-hours, per day. The total time saved is, therefore, not less than 1 000 vehicle-hours per day. By evaluating this time at $2.50 per hour for vehicle, driver, and passenger, extending it over a 300-day year, and capitalizing this amount at 7% to cover interest, depreciation, and maintenance, a total of more than $10 000 000 is obtained. Furthermore, there is the saving to lower-level traffic, which amounts to an average of 1 min. for two-thirds of all the traffic. Using a traffic flow of 15 000 per day, a total of 10 000 vehicle-minutes, or 167 vehicle-hours, per day is saved. Evaluating this at $3 per vehicle-hour, extending it over a 300-day year, and capitalizing at 7%, gives another saving of $2 000 000. It might be argued, therefore, that the city would be justified in spending at least $12 000 000 to obtain a two-level instead of a one-level street over this distance. As shown previously, however, the two-level improvement costs only $6 000 000 more than a one-level street would have cost. This extra cost is all in construction, which is two and one-half times as much for a two-level as for a one-level street. In contrast to the South Water Street project, however, no extra land was required in this instance to make the two levels possible. It is found, therefore, that, for this "Boulevard Link" Improvement, the two-level arrangement is entirely justified.

South Water Street.—This projected two-level street (Figs. 13 and 14) is called the South Water Street Improvement. Its objective is many-sided. Even the one-level improvement originally contemplated would accomplish several important purposes: First, it would remove from South Water Street a badly located, overcrowded, and inefficient Commission Market, the principal market of the entire city. In addition to the street obstruction due to the operation of the Market, the traffic to it and from it is one of the major elements of congestion in the other down-town streets. Second, by raising the street to the elevation of the bridge floors, the present steep ramps from South Water Street to the bridges would be eliminated. Third, this improvement would provide a by-pass around the painfully congested "Loop" streets for traffic going west or southwest, and *vice versa;* also, it would serve to distribute traffic through the "Loop" as from a header or feeder. Fourth, it would offer opportunity for the improvement of dilapidated river-front property and for a monumental boulevard treatment of the water-front itself.

Such a one-level street would have sufficient capacity for all the combined traffic that both levels of the proposed project are likely to receive. In that case, however, as may be seen from Fig. 13,* the South Water Street Improvement could not connect with the upper level of Michigan Avenue and, at the same time, carry away the heavy commercial traffic flowing from the lower level of Michigan Avenue. It is extremely important to have this lower-level traffic carried on South Water Street so as to avoid the "Loop" streets.

* A drawing of the Chicago Plan Commission.

FIG. 11.—VIEW LOOKING NORTH ON MICHIGAN AVENUE IMPROVEMENT.

FIG. 12.—VIEW OF TWO-LEVEL MICHIGAN AVENUE BRIDGE.

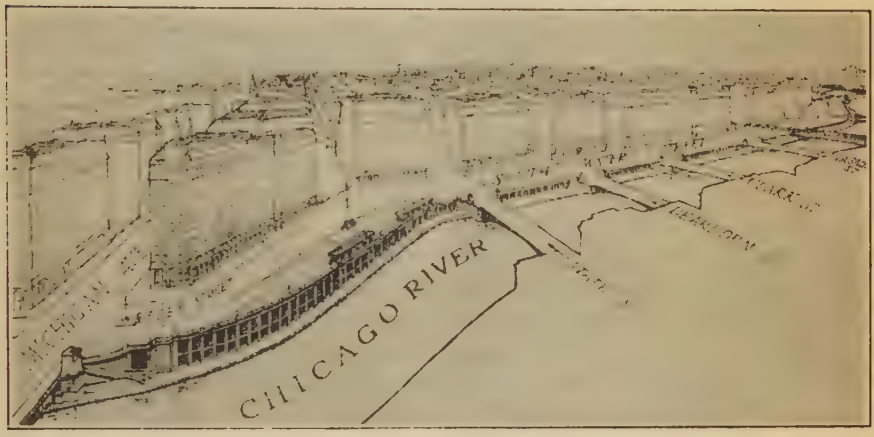

FIG. 13.—THE SOUTH WATER STREET IMPROVEMENT (PROPOSED), EXTENDING WEST FROM MICHIGAN AVENUE.

FIG. 14.—THE SOUTH WATER STREET IMPROVEMENT (PROPOSED), IN CROSS-SECTION.

However, the lower level of the proposed South Water Street Improvement will serve for more than an outlet to the Michigan Avenue lower level. Although it will carry this heavy traffic without interruption at cross-streets over a separate right of way around the congested business center, it will also eliminate the otherwise serious factor of delay caused by crossing the line of each of the upper-level streets passing out of the "Loop". This is particularly important where it crosses La Salle Street, now proposed as a major artery, similar to Michigan Avenue. Finally, the lower level has some value for commercial wharf purposes along the river.

If this lower level along South Water Street was not required to co-ordinate with the Michigan Avenue improvement,* would the two-level structure be justified? It is officially estimated that the two-level street will cost about $22 000 000, whereas the same improvement carried out on one level only is estimated to cost about $11 000 000, or 50% as much. In contrast to the situation at Michigan Avenue, two-thirds of the excess cost is necessary in this case for extra land.

This extra cost of $11 000 000 will separate perhaps 6 000 commercial vehicles per day on the lower level from, perhaps, 10 000, or even 20 000, passenger vehicles per day on the upper level.

It is difficult to estimate accurately the saving in time which will accrue to vehicles by virtue of the two-level separation, but it can be approximated. About two-thirds of the lower-level traffic will continue over the full length of the improvement. The delay, therefore, to this traffic would not exceed, for a one-and-one schedule at cross streets, an average of ½ min. each for one-half the traffic at each of seven cross streets, or a total of 175 vehicle-hours per day. The reduction in average speed from 20 to 10 miles per hour for two-thirds of the traffic would account for an additional 150 vehicle-hours per day. Applying these figures to a 300-day year, and capitalizing at 7%, the sum of about $4 000 000 is obtained. The upper-level cross-street traffic, totaling, perhaps, 30 000 vehicles per day on all the seven streets, would be saved, on an average, not more than ½ min. each for one-third of the vehicles. This produces about 85 vehicle-hours per day and capitalizes to a saving of another $1 000 000. The saving to traffic effected by the South Water Street Improvement does not total more than $5 000 000 or $6 000 000, and this falls far short of the extra cost of $11 000 000 for the two-level arrangement.

Although maintaining that the two-level structure is necessary by virtue of its relation to the upper level of the Michigan Avenue Improvement, nevertheless, it is not believed that, considered as a separate project, the advantages of the two-level structure justify the additional cost. The lack of justification is due to the great cost of extra land needed for the two-level structure, and to the small volume of traffic which can use it.

Considered together, the Michigan Avenue and the South Water Street Improvements, as proposed, will have cost about $37 000 000. As one-level improvements, they would cost a total of perhaps $20 000 000. The total extra

* The plans for the "Boulevard Link" were completed in 1913, before the plans for South Water Street were formulated.

expenditure is not excessively burdensome and is justified because of its vitally important effect on the traffic capacity of Michigan Avenue and of the other streets concerned. This is true, however, only because of the critical importance of the arteries involved; probably it would not be justified for any other situation in the City of Chicago. In addition to the cost, there are always considerations of darkness and poor ventilation on lower levels, and also of the added load placed on other streets to carry the traffic to and from the high-capacity two-level streets, pyramiding rather than solving the congestion problem, as illustrated by the need for two levels on South Water Street to co-ordinate with Michigan Avenue.

There are frequent suggestions for two-level streets throughout the "Loop" in Chicago. A far more rational and desirable solution of the downtown congestion problem lies in decentralization of business and commerce—the process of drawing business and the traffic that serves business away from the over-congested centers, by planning and building the public services with this aim in view. Representatives of both the Chicago Plan Commission and the Chicago Regional Planning Committee have gone on record as holding this kind of decentralization as one of the main objectives of their work—to make the two-level street unnecessary in the future.

HARLAND BARTHOLOMEW,* M. AM. SOC. C. E.—Mr. Corbett's ingenious plans represent as interesting a solution of the problem of separation of various kinds of street traffic as could probably be devised. They are better by far than any actual work that would be done and illustrate how difficult and complex would be the problem of private property development. Their success is almost entirely dependent on co-operation and simultaneous improvement of a large number of adjoining properties—an extraordinarily difficult thing to accomplish.

Assuming that some city might find itself fortunate enough to accomplish and finance such a separation of street traffic, what problem would have been solved? More traffic would be accommodated, but it would be just a question of time before one, if not all, of the several street levels would be congested—and then what? More levels to accommodate more traffic? Where would be the end? This plan is certainly not a solution of the traffic problem; that must be found in a different method of approach.

No city has sufficient street space in its business districts to accommodate all its traffic. The volume of use is increasing, making the percentage of traffic that can be accommodated in the streets of business districts increasingly less. Even though a saturation point of motor-car ownership might be assumed, it would be financially impossible to provide for unlimited accommodation of all kinds of traffic.

The first step toward a solution of this problem is the reduction or elimination of unnecessary traffic movements. Studies recently made in several cities indicated that from 20 to 40% of the vehicle movement was commercial in character. The remainder consisted of individual passenger cars, a small proportion of which conceivably might be engaged in strictly commercial use.

* Engr., City Plan Comm., St. Louis, Mo.

Certainly, a large percentage of present street traffic bears little relation to the business development of the community. Such traffic should either be eliminated entirely from congested areas or at least be confined to a limited number of streets.

The second step toward a solution of this problem is to correct existing street widths and arrangement so as to permit the greatest possible freedom and circulation of traffic movement without inviting prohibitive expense.

The third and last step will be the decentralization of congested areas either by enlarging the area of central nuclei or the establishment of outlying sub-centers, or both.

There can be no complete solution of this problem without the limitation of the size and growth of cities. To invite the tremendous expense implied by Mr. Corbett's plans is to enter on an endless pyramiding of expense. The idea of constantly increasing the height of buildings and attempting to provide accommodation for an ever-increasing traffic at their base is making progress backward and can lead only to economic strangulation.

The two instances of separated street grades in Chicago cited by Messrs. Hill and Crane are excellent illustrations of the economic impossibility of the universal application of Mr. Corbett's plans. There are not ten cities in the United States to-day that can afford rapid transit subways, for instance, and yet hundreds of cities have street traffic congestion problems that are becoming more and more acute, and that require immediate relief.

The fact that separated street grades will solve no traffic problem permanently is justification for the assertion that proponents of these should not look to public revenues for their execution. Before inviting huge public expenditures for work of this character, engineers should first ask how far a city's credit might be taxed for the purpose of securing increase in street capacity. When the huge demands already made on municipal revenues and credit are considered, it becomes increasingly clear that any city's financial ability to increase street space is extremely limited.

To repeat—the solution of the problem of street traffic congestion lies not in doing those things that invite greater concentration, but in doing those things that will lessen or distribute traffic, such as (1) reducing unnecessary movements; (2) correction of defective means of circulation; and (3) enlargement of area, reduction of height, and decentralization of business districts.

T. Kennard Thomson,* M. Am. Soc. C. E.—The Society is greatly indebted to Mr. Corbett for his valuable and remarkably well presented discussion. For more than thirteen years, the speaker has been working on a problem which is much simpler, inasmuch as Mr. Corbett's task is hampered by existing conditions, whereas that of the speaker is untrammelled by such obstacles. There is an opportunity to create an idealistic city 9 sq. miles in area, in the heart of New York, without destroying any existing property, simply by extending Manhattan 6 miles down the Bay.

Mr. Corbett's excellent suggestion that cross streets pass above longitudinal streets would certainly be a great relief to the present congestion.

* Cons. Engr., New York, N. Y.

W. W. Crosby,* M. Am. Soc. C. E.—The ideas suggested have been extremely interesting.

In Los Angeles, Calif., where the speaker has been frequently, traffic conditions have improved, but are not yet such as to set an example for New York. The excellent suggestion made in regard to a platoon system of regulating traffic, has some defects in practice, as pointed out by Mr. Bartholomew, and those defects are decidedly in evidence in Los Angeles at present.

It must be acknowledged that, particularly in America, rules and regulations for pedestrians are difficult to enforce satisfactorily. Automatic or subconscious control works better, provided the automatic regulator is not too delicate or too likely to get "jammed." The great trouble in Los Angeles is the habit of pedestrians to ignore signals, and the tendency of other traffic to defy them.

The theory of separating foot and vehicular traffic appeals strongly as a matter of simple automatic control. It may be of interest to note that one of the oldest cities in England—the City of Chester—has double-decked arcaded ways for foot passengers, which would illustrate Mr. Tuttle's ideas. There, however, both stories are still used for foot passengers and the automobile has not yet appropriated the lower level for parking spaces.

The arcaded streets in Europe and elsewhere are well known and have succeeded in greatly increasing the vehicular capacity of the roadways.

In New York, an original through street might be compared to a feed-wire to which an excessive number of outlets have since been added. The question now becomes one of keeping down the additions or of enlarging the feed.

It may be remembered that in 1884 there was a proposal to build a subway under Fifth Avenue which was thus to be made a double-deck street and an important "feed". The problem in New York has been attacked and partly solved, but it seems that some repression of the demands on the streets will have to be enforced or else some suggestion such as that of Mr. Corbett will have to be adopted.

George S. Davison,† M. Am. Soc. C. E.—The word "repression" has been used in this discussion. The speaker thinks there has been, and, in the future, there will be, a great deal done in the way of repression that will help the condition of congestion in large cities. Other discussers have evidently in mind that the pleasure car business will grow, and, consequently, its traffic will continue to increase at the present rate. The speaker does not share that idea; he thinks there will soon be a slackening of this business as it affects congestion, and that finally the point of saturation will be reached. It does not seem possible that the automobile industry can keep on making cars at the present rate, to be absorbed by new customers in the United States. The manufacturers will work largely on substitutions for worn-out equipments, which naturally will not increase congestion. The demand for cars by people who have not previously owned them, must diminish materially; otherwise, the time would come when every adult would own and operate a car, which con-

* Cons. Engr.; Location Engr., State Dept. of Highways, Harrisburg, Pa.

† Pres., Gulf Refining Co., Pittsburgh, Pa.

dition is not conceivable. So, generally speaking, the problem of providing for the future automobile traffic is not as serious as stated.

The problem in New York is different from that of Pittsburgh, Pa., where the speaker is a member of a committee which has recently been appointed to study the problem of street congestion. As it would appear at present, Pittsburgh cannot afford to have rapid transit subways; the best it could do would be to build subway loops for trolley cars in the congested part of the city. Building such loops, would relieve the congestion of the traffic in a small area known as the "Golden Triangle", but even then only by a few hundred street cars each twenty-four hours. This would not materially reduce the congestion, which in Pittsburgh, as elsewhere, arises through the use of the streets by thousands of pleasure automobiles and commercial trucks. Not the least of the difficulties arises from the parking of cars in double line along narrow business streets, thus retarding the flow of traffic on such sections of the roadways as are not occupied by these standing vehicles. The large department stores are located within this area, and the "lady of the house" adds her share of confusion by causing her car, which may represent only herself, or, at most, a family of three or four, to occupy 100 sq. ft. of space for an hour at a time. Meanwhile, the street car carrying 40 or 50 people, is occupying a moving area of not to exceed 400 sq. ft.

As far as the Committee has observed considerable automobile and truck traffic can be by-passed around the "Triangle" without any serious inconvenience, and those who now drive their cars into parking spaces within it can, with no additional discomfort to themselves, leave their cars at parking spaces at the edge.

There is an economic question involved in this whole matter, at least in Pittsburgh—the large sums of money which it may be necessary to expend for underground loops for street cars must have their interest payments carried for many years and eventually be amortized. Who shall be burdened with these payments? Surely not the street car rider, because he is now complaining, and with some reason, that he is paying too much at present. It is not his vehicle that is the primary cause of the congestion. If not he, then it must be the taxpayer. Arriving at that conclusion, it must be considered that comparatively few of the tax-paying class are involved in the reduction of congestion; these few are the automobile owners.

It would seem that there are two general plans on which to work. One would be to formulate better regulations for controlling the automobile and truck traffic in order that the occupation of the streets might be greatly reduced; and the other, planning street improvements for by-passing purposes, the automobile and truck owner carrying the cost. The speaker realizes that there must be new legislation enacted to cover the latter case, but believes there is fully as much reason for the vehicle owner to pay a license fee to the city wherein he resides, as that he should pay a fee to his State, from which his benefits are much more obscure than those that would inure to him in the maintenance of proper traffic conditions in his city.

GEORGE T. SEABURY,* M. AM. SOC. C. E.—The idealistic plans presented by the author have been inspiring and some of the vision has been caught, but, in the main, they do not provide the practical solutions that are urgently needed at this moment by nearly every city of this land. Traffic problems are imminent and must be solved without delay and without considerable expenditure. Engineers may dream visions; but they also must work out practical simple solutions and at once.

Like Mr. Davison, the speaker is one of a committee charged with devising a solution, applicable at once, to the congested existing streets of a busy city, namely, Providence, R. I. Trolley, vehicular, and pedestrian traffic is now using all the pavement not occupied by standing automobiles and all too frequently it finds the space insufficient.

It is evident that any practical solution must be based on the premise that most of the vehicles that wish to use the pavement area for purposes of transit must be allowed to do so. If the premise were that all vehicles that desired to move through the streets of the congested area should be allowed to do so, the most obvious solution would be the elimination of parking. It is not necessary, however, for all vehicles to use those streets and segregation may be made between streets primarily for trolleys and other streets without trolleys. It is practicable to re-route either the automobiles in large part, or the trolleys. In Providence, the latter has been accomplished recently, though with loud complaint—not entirely unjustified—from the trolley users. Admitting the possibility of re-routing even more, if necessary, the solution must depend on some other factor.

Providence is not so laid out that it suffers particularly from through traffic; at least, none of the streets that now need attention needs it for that reason except during the afternoon rush hours. Consequently, any attempted segregation of vehicles reduces rather to segregation of types than to segregation on separate streets. The mind at once jumps to the tentative decision that vehicles used for pleasure or, perhaps, even for convenience, should not be permitted in the center of the city. Such a conclusion is somewhat warranted by an analysis of the amount of space used by the several types of vehicles as compared with the justifiable use made of that space. It appears that the trolley car is used to move about 80% of the population that does business in the congested area and that the remaining 20% is moved by automobiles. It appears, moreover, that one trolley renders service to approximately 40 people, whereas one automobile serves only about 2 people; in other words, a trolley is equivalent in carrying capacity to 20 automobiles. The street space needed by a moving trolley is approximately 100 lin. ft., whereas the space on the same street needed for a moving automobile is from 20 to 25 ft. Therefore, the use of street space for the transportation of people by trolleys is more efficient than that by automobile in a ratio of at least 4 to 1. It is admitted that subways for trolleys would be advantageous, but subways are an impracticable solution on account of both time and money.

It has been shown that as between the trolley and the passenger automobile, the latter must yield; this conclusion is strengthened by the necessity of the

* Secretary, Am. Soc. C. E., New York, N. Y.

use of the pavement by automobiles engaged in other than passenger traffic. It would inflict an almost impossible burden on the transaction of business to exclude commercial vehicles from the use of the streets not only for purposes of transit, but for loading and unloading. Here, again, the elimination of the parked passenger automobile seems the logical immediate solution. It is admitted that the commercial vehicle abuses its logical pre-eminence. The towel-supply truck, the ice cart, the spring-water delivery automobile all loiter along, and too frequently stand unused in front of a business house for long periods. In the main, however, the city is for business and the means of doing business must be given preference.

The much-to-be-encouraged business, however, does not confine itself to the use of any one type of automobile. Any general segregation based on the type of vehicle is, therefore, inapplicable to the great majority of streets. Thus, one is largely at sea as to a solution based on the foregoing line of reasoning. The premise in regard to the necessity of making the present streets adequate for the use of those vehicles that find it necessary to move in them is all that remains.

Granting this premise and laying aside, for the moment, any opinions with regard to segregation of vehicles, etc., it seems there is a constructive method that should lead to a solution of the problem, as illustrated by the plan the Committee is following in its study in Providence.

Considering each street by itself, an attempt is being made to decide how many lanes of moving traffic are needed to permit the passage of those vehicles that require room in that particular street. Coincident with this, a decision is made as to whether or not it is possible to permit the standing of automobiles on that street. When the width of the existing pavement is such that the requisite number of moving lanes can be secured and, in addition, automobiles of either the commercial or passenger-carrying type may stand parked, well and good. On the other hand, if this is not possible, it is proposed to consider that the passenger type of vehicle may do no more than stop to take on or to let off passengers, nor the commercial vehicle do more than stop for the time necessary to load or unload with reasonable dispatch merchandise ready for loading or unloading.

By such a process of elimination it will undoubtedly develop that the permissible parking space will be curtailed. With ever more vehicles desiring to park and with less space in which to do so, the only equitable solution (unless adequate and convenient parking spaces in addition to the pavement are provided) seems to be a reduction in the time during which parking may be permitted. This solution, it is believed, will not of itself result in a completely satisfactory use of the streets throughout the entire day. The number of moving lanes needed will vary with the hour. It is proposed, therefore, to base the number of moving lanes needed on the early afternoon traffic and to designate streets or zones in which, during the late afternoon rush hours, all vehicles shall be required to move. In this "all-rolling zone", both the parking of automobiles and the loading and delivery of merchandise will be eliminated except in emergency or by permit.

E. P. Goodrich,* M. Am. Soc. C. E.—It has been interesting to have had the economic element brought into the traffic problem. Modern engineering is a question of economics, fully as much as of the physical creation of things for the benefit of man.

In somewhat the same way that Messrs. Hill and Crane have calculated the saving due to double streets, estimates have been made of the present cost of congestion in various cities. Offsetting those congestion costs, the costs of improvements have been determined, thus placing the whole problem on an economic or financial basis. It has been estimated that Cincinnati, Ohio, loses $100 000 per day due to congestion. Not all of this, however, if capitalized, would be a proper expenditure for removing the physical difficulty.

The best practice is to make use of existing roadways to the utmost. Working along that line a scheme was devised, some years ago, which might be described as the platoon system of operation. For example, imagine a company of soldiers marching northward on Fifth Avenue, in New York; following it is another company with a gap between, succeeded by a third platoon. A similar series marches down the street. Obviously, where the gaps meet, cross traffic can operate. By proper spacing, it has been found that a system of rectangular streets could be co-ordinated or synchronized so as to keep any particular vehicle moving continuously in any direction without stops, the groups of such vehicles interlocking at the cross streets.

It can be illustrated in another way. All engineers are familiar with the time space diagram. Assume the vertical ordinates as distances along Fifth Avenue; at appropriate points are the cross streets; time is plotted horizontally. Therefore, a vehicle starting at a certain point and moving up the Avenue, would be shown graphically by a diagonal line. A subsequent movement of vehicles would be represented by a diagonal displaced to the right. Those coming down would be represented by lines sloping in the other direction. On any cross street, the same scheme could be operated, if properly arranged.

A somewhat similar scheme has been in operation in Los Angeles, Calif., for some time, traffic in parallel streets being operated and stopped in a synchronized manner, so that in every alternate street, it moves alternately. A system of semaphores, somewhat like the block system mentioned by the author, is in operation. Such a system would increase the efficiency of the present Fifth Avenue operation by 100 per cent. The Los Angeles system is far from perfect but is a step in the right direction.

It may hearten Mr. Davison who has discussed the situation in Pittsburgh, Pa., to hear that in practically every State in the Union the number of automobiles is approaching the saturation point. Doubtless the traffic will correspond approximately with the ownership of automobiles. That has been proven in different districts where traffic observations have been made over long enough periods.

Mr. Bartholomew put his finger on an important point when he mentioned the elimination of parking. Many times that question has been raised, but it is yet to be answered satisfactorily.

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* Cons. Engr., New York, N. Y.

JOHN A. MILLER, JR.,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper treats of a subject of great interest and importance at the present time and contains a number of good suggestions for the relief of traffic congestion. However, the removal of tracks from the streets and the substitution of busses for the electric railways, a plan which the author seems to favor, would greatly increase congestion instead of relieving it.

He asserts that the economic advantages of the surface car over the bus in street capacity used have been lost to a considerable extent in crowded sections of the city because of the slowing down of speed. Experience in other cities where large numbers of busses have been operated, does not confirm this assertion. On the contrary the superiority of the street car over the bus is most evident where congestion is the greatest.

In order to reduce the cost per passenger-mile of motor-bus operation to approximately that of electric railway operation, bus development in the United States has been confined almost entirely to vehicles operated by one man. This places a restriction on its carrying capacity, which may vary from 30 to 50, depending on the degree of crowding which is permitted, but will not ordinarily exceed 50 as a maximum. Under the same conditions of loading, a street railway car can carry from 100 to 150 passengers. It is evident, therefore, that there would have to be three or four times as many units on the street to carry all the traffic by bus. The resulting congestion would be much worse than the present condition.

Extensive investigation made by the writer over a period of years developed the fact that in congested districts, with busses and cars operating side by side on the same streets, the bus was from 10 to 20% faster than the car. This superiority in speed resulted primarily from the fact that the bus was making no attempt to handle all the traffic, but was simply providing a sort of express service for the limited numbers of persons whom it could accommodate. Time was saved because the busses received capacity loads at or near the beginning of the route, and proceeded without making additional stops. From this, it is clear that the greater speed of the bus is the direct result of its smaller carrying capacity.

The 10 to 20% superiority in speed, however, is not enough to offset the 200 to 300% increase in the number of units which would be required if busses were to carry the entire traffic. Although the use of double-deck busses would reduce somewhat the number of units needed, their greater carrying capacity would increase the number of stops required, and this, coupled with the delays incident to passengers climbing up-stairs and down-stairs, would reduce their speed to a rate somewhat slower than that of the street car. It appears, therefore, that the theory of lessening traffic congestion by the substitution of busses for street cars is not sound. Moreover, mass transportation by bus is unquestionably far more expensive than by rail.

In his discussion, Mr. Corbett states that wheel traffic is gradually absorbing that formerly carried by rail, because more people are riding in motors and fewer on street cars and in subways. Statistics do not bear out this contention. The figures of the Transit Commission (Table 15) show more pas-

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sengers being carried by the railways in New York every year, the number in 1923 being approximately 14% greater than in 1920.

It is very easy to state, as Mr. Corbett does, that all rail traffic should go underground. However, in view of the constantly increasing traffic on all the railways, how can sufficient subways be built quickly enough to permit the removal of surface and elevated lines? No one has yet been able to suggest a method.

TABLE 15.—Revenue Passengers Carried in New York, N. Y.

Year.	Surface lines.	Rapid transit lines.	Total.
1920	941 420 788	1 331 915 745	2 273 336 533
1921	977 652 062	1 418 649 471	2 396 301 533
1922	1 052 968 891	1 438 239 919	2 491 208 810
1923	1 071 736 876	1 506 076 001	2 577 812 877

As neither the substitution of busses nor the building of more subways is a feasible method of handling the passengers now carried on the surface, railway tracks must remain in the streets. It would be possible, however, to increase the capacity of existing streets by speeding up the movement of traffic. The present slow speed of surface cars which the author uses as an argument against them, is to a considerable extent the result of conditions that can be remedied. If traffic were regulated so that the tracks were kept clear for the cars and not obstructed by motor and horse-drawn vehicles of every description, and if traffic officers were encouraged to make a greater effort to coordinate their signals with the movement of cars, a substantial increase in speed for all traffic would be possible with no expense to any one.

It would be very unfortunate if the public became imbued with the idea of tearing up the car tracks in New York before any adequate substitute has been found to handle the tremendous traffic. Perhaps such a step is not actually contemplated, but the frequent talk to this effect, whatever its purpose, is creating a feeling of doubt and uncertainty in the public mind which is working a great hardship on the railways and seriously interfering with their program of reconstruction and improvement.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—The problem of increasing the capacity of streets is one of interest not only to great cities, but to small towns as well. In many of the latter serious congestion is now being experienced, especially in those through which pass great State and National highways.

In Southern towns the streets are quite generally narrow, from 25 to 30 ft. between curb lines, and sometimes even less. In Colonial days, these widths were ample and remained so until the advent of the automobile; in fact, no serious trouble was experienced until recently. Local motor cars have multiplied, but more important in some respects is the great surge of State and interstate traffic, creating a condition which the citizens in their own business interests and for their own comfort and protection cannot ignore.

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The authorities of many Southern towns on the Lee, Bankhead, Dixie, and Southern National Highways have realized this and are now seeking relief in police regulation of traffic, restricted parking, and the entire elimination of parking. At first, parking on one side of the street is prohibited; then parking on both sides. Finally, the merchants come to a realization of the effects of these regulations on their business and clamor for the removal of the parking restrictions. Evidently, however, such a return is impossible unless a drastic procedure is adopted—either the widening of the street or the restriction of traffic to one direction. To those who seek a profit from the tourists, the latter solution is unsatisfactory.

In addition to the local interests, however, there are also great outside interests of State and National character. The State Highway authorities realize the throttling effect of towns through which the highways pass and, therefore, can be expected to co-operate with the local authorities in re-location work and bridge construction when the benefits belong more to the State than to the locality.

Specifically, the Town of Lexington, Va., the resident population of which is 3 000, will be taken as an example. It is a university town, the seat of two institutions of learning and the burial place of Generals Lee and Jackson. It is often spoken of as the "Shrine of the South"; gauged by the volume of incoming and outgoing motor traffic, it is indeed becoming more and more so. Until recently this traffic was inconsiderable; but it is now increasing rapidly due to the development of the Lee Highway through Virginia, now nearly completed. A heavy burden is thus thrown on streets which were intended to serve very different purposes. The State Highway Department cannot be expected to assume the expense of street widening; local interests must work out their own salvation. In the center of the town, the highway measures 30.6 ft. between curbs and 50.7 ft. between building lines.

A plan has been prepared to widen the street to the face of an important bank building which was built 6.15 ft. back of the building line, thus giving a total width of 56.85 ft. between building lines and 38.85 ft. between curbs, the removal of poles on both sides of the street providing an extra 2.1 ft. between curbs. Of the cost of this improvement, 60% should be assessed as benefits on abutting properties, the remainder being provided from the general taxes.

A second solution of the problem is to provide for double routing between points of junction, no street widening being necessary. This is possible by reason of the existence of two parallel streets through the town, 300 ft. apart, that meet at an acute angle at the northern end of the business district, one of them being the main street. At the southern end of the town, it would be necessary to create a similar junction by a re-location of one street and the removal of one or two houses. This solution presents less obstacles and could be effected at small expense. It would not prove satisfactory to the business interests of the town as they now exist, however, because it would eventually result in the transformation of the street that is now partly undeveloped and

partly residential, to one lined with business houses catering to the needs of tourists.

In either case, however, for at least 0.8 mile, the approach to the business district would have to be widened, as the roadway is only 22 ft. between curbs and inadequate even at the present time with the Lee Highway still uncompleted. Traffic censuses show great overcrowding with the prospect of a continuing increase. To provide for it, the street should be widened to 26 ft. between curbs, with a re-location of the highway in the northern end of the town to eliminate dangerous curves. A new bridge with approaches will also become necessary, a part of the cost of which should be borne by the State.

Under the first plan, space is provided in the heart of the town for parking cars from both directions in the middle of the street at an angle of 45° to the curb. This treatment is believed to be unusual, but has much to recommend it. With the second plan, by which double routing is proposed, ample parking space on one side of each street is provided. The roadway width of 26 ft., previously mentioned for the northern approach to the point of junction, will provide for parking space on one side of the street with ample clearances for cars traveling in opposite directions at fairly high speeds.

In order to save expense, set-back lines on the main street have been considered; eventually, they would achieve the desired result of street widening, as new buildings could not be constructed in front of these lines, nor could reconstruction be carried beyond them. This plan evidently would involve great delay in effecting the result desired and is not favored for that reason. If a plan for immediate widening is adopted, the arcading of certain buildings might seem desirable owing to their historical value.

Lastly, there is the troublesome problem of assessments for benefits. In Virginia, assessments of this kind can probably not be levied under the amended Constitution of 1902, which reads:

"No city or town shall impose any tax or assessment upon abutting land owners for street or other public local improvements, except for the making and improving the walkways upon then existing streets, and improving and paving then existing alleys, and for either the construction, or for the use of sewers; and the same when imposed, shall not be in excess of the peculiar benefits resulting therefrom to such abutting land owners."

Evidently, this section is in need of further amendment.

A. C. Janni,* M. Am. Soc. C. E. (by letter).—This timely paper is very apropos of the transportation problem of Lower Manhattan. It is axiomatic that the traffic problem in New York is characteristic of New York; therefore, it should be solved with respect to the peculiar physical shape of the city and with the knowledge of the exigencies of its business life. The general methods of handling traffic in London and Paris, as well as in some of the larger American cities, although useful as far as the study of their minor details is concerned, cannot be taken as models for other cities where conditions are altogether different.

It is said that in the past few years the traffic in the lower part of New York has increased about 100%, and everything points to a progressive growth

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year by year. There are two factors in the question which, so to speak, work against each other, namely, the physical conformation of this district and its enormous importance. It is true that part of the congestion is gradually being shifted toward 42d Street, but, thus far, this has not sensibly relieved the condition on account of the constant increase in the population and in the importance of New York with respect to the other markets of the world.

Ways and means for relieving this congestion have been studied and proposed. Some of them are merely makeshifts, not worthy of a metropolis like New York; some are rather expensive, but inadequate; and a few have taken into due consideration the main source of difficulty in this problem.

A map of part of New York (Fig. 16) shows that in Lower Manhattan the north-south transportation is fairly well developed, but the east-west facilities are almost completely neglected; this is one of the causes of the highly congested traffic on the main east and west arteries in this locality at rush hours. The rapid transit lines are congested on the backbone of the island, so that pedestrians are bound to become crowded in reaching their means of transportation. Furthermore, the elevated lines are located on streets which would be excellent north-south arteries of traffic, were it not for the interference of these structures. Finally, for the development of New York toward the north and northeast of Manhattan, adequate long hauling express transportation is lacking.

From 110th Street north the transportation between subways and elevated is well distributed, except for some additional cross-connection lines that probably would help it greatly; but this, at least for the present, is not a crucial deficiency. Naturally, in adopting any new transportation plan, the trunks of the elevated lines north of 110th Street should be equipped adequately for their carrying capacity south of 110th Street; that is, the construction of these trunks should be changed to correspond with the new exigencies of transportation.

It is not a precipitate judgment, therefore, to say that the elevated transportation as it is located at present, sooner or later must disappear, to give place to something which fulfills the same task in a more logical and far less obtrusive way. In other words, this type of elevated line must submit to what has already happened in all progressive cities, to the wiring for the telephone, the telegraph, and the trolley car.

It will be seen from Fig. 16 that the new system of transportation proposed by the writer contemplates:

1.—The retention of all present subways, for they are well located and accomplish satisfactorily their work of transportation.

2.—The removal of all present elevated lines from South Ferry to 110th Street.

3.—The substitution of two trunk elevated lines. One should start at South Ferry and run along the Hudson River front to 110th Street, there connecting with the old elevated line (this line, the west line, should have six or eight tracks for expresses and local trains); the other could run from South Ferry to 110th Street along the East River front, as shown, connecting at 110th

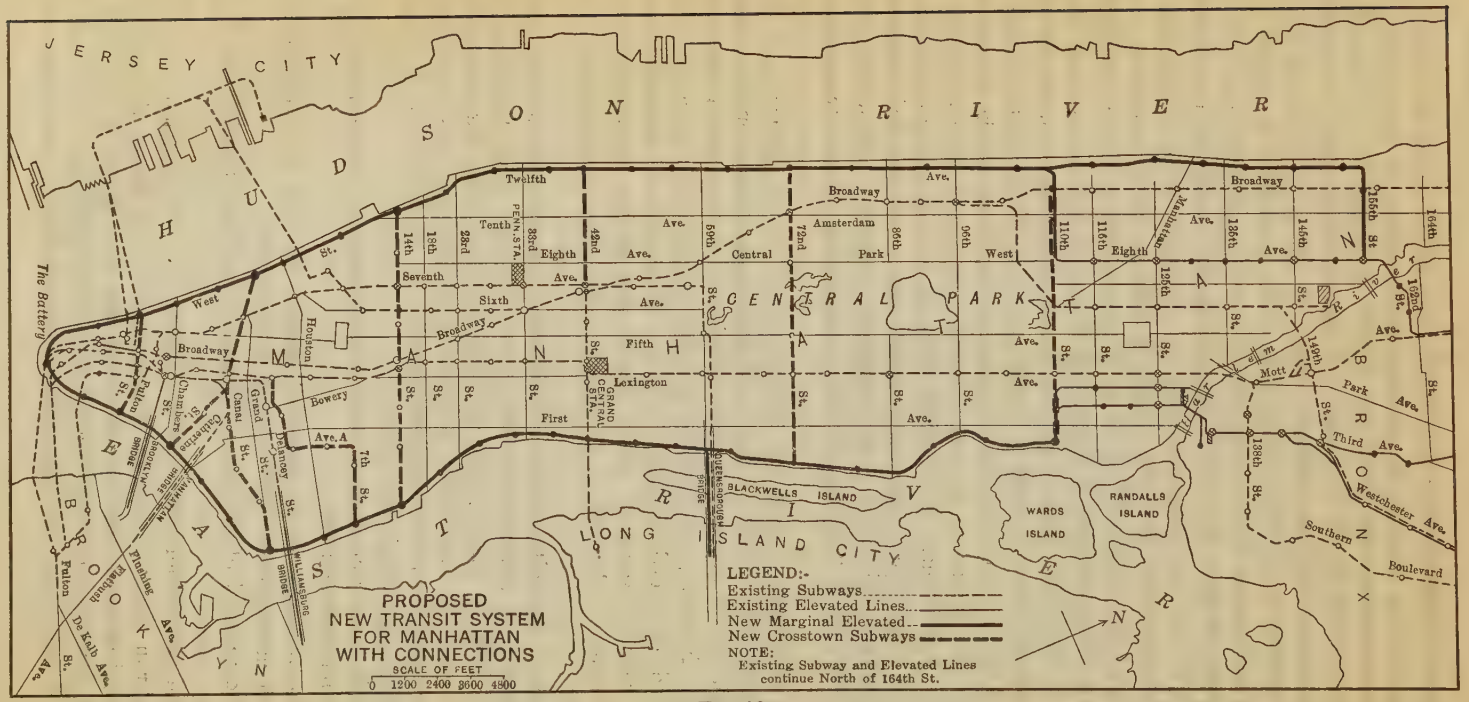
JERSEY CITY
HUDSON RIVER
The Battery
Broadway
West
Houston St.
Bowery
Canal St.
Grand St.
Delancey St.
Ave. A
7th St.
Fulton St.
De Kalb Ave.
Flushing Ave.
Williamsburg Bridge
Manhattan Bridge
Brooklyn Bridge
Pearl St.
Chambers St.
Park Row
Twelfth Ave.
Tenth Ave.
Eighth Ave.
Seventh Ave.
Sixth Ave.
Fifth Ave.
Broadway
Lexington Ave.
First Ave.
14th St.
18th St.
23rd St.
28th St.
PENN. STA.
42nd St.
49th St.
50th St.
59th St.
GRAND CENTRAL STA.
Central Park
72nd St.
86th St.
96th St.
110th St.
116th St.
125th St.
136th St.
146th St.
155th St.
164th St.
CENTRAL PARK
Amsterdam Ave.
West End Ave.
Eighth Ave.
Lenox Ave.
Seventh Ave.
Broadway
Mott Ave.
Park Ave.
Third Ave.
138th St.
Westchester Ave.
Southern Boulevard
BRONX
BROOKLYN
EAST RIVER
LONG ISLAND CITY
BLACKWELLS ISLAND
WARDS ISLAND
RANDALLS ISLAND
PROPOSED
NEW TRANSIT SYSTEM
FOR MANHATTAN
WITH CONNECTIONS
SCALE OF FEET
0 1200 2400 3600 4800
LEGEND:-
Existing Subways
Existing Elevated Lines
New Marginal Elevated
New Crosstown Subways
NOTE:
Existing Subway and Elevated Lines
continue North of 164th St.
N

FIG. 16.

Street with the old elevated system (this east line could have from six to eight tracks likewise).

4.—A system of cross-connecting subways which would make it possible to transfer to all lines, both elevated and subway, as needed. These changes should be made possible on the basis of a single fare.

It seems desirable that the east and west express lines should have few stations south of 14th Street; and that from 14th Street to 110th Street there should be no stop whatever, so as to make possible the transportation of passengers living north of 110th Street, thus relieving the traffic in Lower Manhattan as soon as possible. It will be seen, also, that the connecting cross subways would suffice to carry people from the center of Lower Manhattan to its margins, and *vice versa.*

Furthermore, in order to get to the proposed elevated, passengers will either disappear from the surface through the stations of the longitudinal and cross-connecting subways, or will walk to the east and west elevated lines. Thus, the great number of pedestrians will be spread over a far larger surface than at present.

The cross subway connections at Fulton, Canal, and 14th Streets will then accommodate practically all the pedestrians, thereby avoiding the great congestion on these streets, which is now intensified in rush hours. For that matter, the entire traffic of Lower Manhattan, as far as pedestrians are concerned, will diminish very materially on the surface of the streets.

The rapid transit lines might be of great assistance to shipping during the non-rush hours, nights, and holidays, if a plan could be effected whereby some of these tracks might be used by railroad companies. Further, by an agreement between the City and the New York Central Railroad Company, the latter might remove its tracks from Tenth and Eleventh Avenues, in exchange for a track of the elevated line, thus removing from the streets a constant source of danger.

The abolishment of the old elevated system would enlarge several thoroughfares, which, at present, cannot be used to their full carrying capacity. For instance, on Sixth Avenue, south of Central Park, by removing the elevated structure, and by cutting, say, 2 ft. from each sidewalk, the resulting roadway would amply carry three rows of automobiles each way; at present, often there is only convenient space for two rows, one each way. Real estate values would also improve materially, and first-class construction like that on Madison Avenue, or even on Fifth Avenue, might result. A similar transformation might happen on Ninth, Third and Second Avenues, should the traffic be altered materially.

A city pre-eminently commercial and industrial is not necessarily ugly and oppressive. Assuredly, commercialism and utilitarianism must have their places, but after all they are not the only considerations in the life of a city. Instead of the present unsightly, noisy, cumbersome, and obsolete structure, the type of this proposed new elevated system should follow quite modern lines. Steel construction, in general, has little claim to being called æsthetic, and its cheaper initial cost is often misleading. By using reinforced concrete the main-

tenance would be a minimum, noise would be materially reduced, and, finally, a pleasing effect could be reached by a few cleverly designed lines.

The river boundaries, like similar parts of all sea towns, never will be residential or shopping districts, where an elevated structure would be highly objectionable. The section of the western elevated structure from, say, 72d Street north, could be lowered to such a level as to become almost a subway with respect to Riverside Drive, and other minor difficulties which may arise in a general plan like this could likewise be overcome. The idea of a marginal elevated railway is not vital to this plan; a marginal subway with the same number of tracks, apart from its financial aspect, would serve just as well. The principal need is to relieve the concentration of traffic by spreading it over the whole surface of Manhattan Island.

The foregoing improvements together with the re-arrangement of the surface car service, both longitudinal and cross-town, would result in a system which would hold good for all time. It may be noted further that the adoption of this plan would not interfere in any way with the present system of transportation while the changes were being made. The solution of this problem of transportation in New York does not involve complex engineering, but merely common sense.

C. E. Grunsky,* Past-President, Am. Soc. C. E. (by letter).—The American people have not yet learned to plan for the future as well as for immediate requirements. Fifty years hence is a long time ahead. Let posterity solve her own problems; why should the present generation worry over the troubles of the next? A little reflection should make the reason for looking ahead apparent. The fifty-year period seems long in the short span of a human life, but it is only a short period in the life of a city. Had our forefathers been blessed with better foresight they might have provided many necessities and conveniences which are now difficult to secure. Perhaps no better illustration can be found than the present-day need of greater street capacity in American cities.

There is an occasional exception to be sure as in the case of Washington, D. C., where many wide streets were provided without in any substantial way depriving the lot owner of what he would have enjoyed had his lot been extended somewhat and the street made narrower. The plan there followed is one that can be recommended wherever new sub-divisions are planned and particularly in cases where a set-back line from a street of ordinary width would be desirable. According to this latter plan the street is laid out extra wide, 120 to 150 ft., and then the improvements are made as if it were only 60 to 80 ft. in width. To each lot owner is reserved the privilege of using the extension of his lot to the sidewalk as if it were his own garden plot. Under such a plan the setting back of sidewalks to widen the roadway can be undertaken whenever desirable; furthermore, the city retains control of the tree planting and parking along streets, thereby being in position to add greatly to their attractiveness.

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The problem of increasing traffic facilities after property has been highly improved is much more complex and difficult than planning with proper foresight when acre property is being sub-divided. The widths of streets in American cities are not adjusted to the requirements of a skyscraper district. As municipalities have now blundered along to the point where every lot owner counts on his right to go high into the air with his building, regardless of the light and sunshine which he takes from his neighbor, the question is well asked as to what shall be done about it and Mr. Tuttle's paper is timely and valuable. Toward the solution of the problem the writer makes the following contribution embodying a suggestion which would appear to be quite feasible in many cases, although, of course, requiring legislation before it can be applied. When the suggestion was first made some years ago he had in mind the widening of Eldorado Street, Stockton, Calif., a north and south street, now 80 ft. wide, but which should be at least 120 ft. wide. The widening of this street or of a near-by parallel street will some day be imperative. Why not acquire the necessary land by purchase at once so that future improvements will adjust themselves to the wider street? If the city had the authority it could effect the acquisition of the necessary land, with the understanding that the present owner would be left in possession for a fairly long term, say, 25 or 30 years. The owner would be paid the market value of the property or, possibly, any reasonable amount which he himself might fix as adequate compensation, but with the understanding that he take from the city a lease to yield 6% (in some cases, perhaps, as high as 7%) per annum on the amount which is paid to him for the property.

Such an arrangement would give an owner 25 to 30 years, as the case may be, to adjust his business to the new street lines. His remaining property would appreciate in value by reason of the street widening. He could, of course, use the funds obtained from the city just as if it were a loan by a bank at the fixed interest rate. He would no longer pay taxes on the property acquired by the city; on the other hand, the city which can borrow money on a bond issue at 4.5% to 5% would be obtaining from its leased property 1% to 1½% per annum more than the financing of the proposition costs, and this excess would materially reduce the actual cost of the street widening when finally undertaken. Where property owners are unwilling to lease back from the city, the alternative would still remain of purchasing and taking immediate possession if a fair price can be agreed on, or deferring the purchase, with possible ultimate recourse to condemnation proceedings. It is believed that owners generally, particularly in down-town fully improved areas, will readily see the advantage of the sale at a good price with long-time undisturbed right to remain in possession as a tenant. Under such an arrangement they would be assured of full realization on the present or remaining value of their improvements.

The acquisition of property long in advance of the time when it is to be taken over might also be applied in another way to improve the facilities which the down-town street affords. Again, looking ahead, it would seem quite feasible to acquire by purchase at once a right of way for sidewalks inside the property line, to be taken possession of 30 years in the future. In this case, the idea is to acquire an easement for a passageway about 20 to 25

ft. wide and 14 to 16 ft. high. This, too, under such a plan would be bought for cash, generally at the lot owner's or owner's and tenant's price, subject to the understanding that thereafter rental would be paid to the city at (about) 7% per annum of the amount paid for the easement. Even the modern building, or at least the part of it that would thus have to be abandoned to the public, can readily be amortized in a 30-year period. If devoted to a high type of use, the price asked will be high, but, as an offset, the rent obtained by the city on, say, a 7% basis, will be correspondingly high. The margin between 4.5% or 5% interest on the improvement bonds and 7% (calculated at 4%) would completely amortize the cost to the city in 24 to 28 years, after which time the easement would stand on the books as having cost the city nothing. Meanwhile, as in the case of an outright purchase of property and release to the original owner, there would be no change for a long period in the use of the property, and at the end of this period the owner would obtain all the benefits of the arcade arrangement, as so well pointed out by the author.

Arthur S. Tuttle,* M. Am. Soc. C. E. (by letter).—Mr. Corbett points out that, as New York grows upward, additional street capacity must be provided to carry the corresponding increase in pedestrian and vehicular traffic. His method of securing relief is based on the adoption of the arcade principle, but is carried much further than that proposed by the writer and his argument in its support is regarded as an endorsement of the practicability of introducing arcades without sacrificing the appearance or attractiveness of the arcaded buildings.

The Chicago improvements, discussed by Messrs. Hill and Crane, were designed to meet special conditions and have undoubtedly served their purpose well. The means proposed for meeting the traffic congestion problem are based mainly on decentralization, which basis is also urged by Mr. Bartholomew. It is often difficult, however, to apply this principle without running counter to the natural tendency of the various kinds of business to herd together. To throttle this trend by drastic means, would be a long forward step toward traffic relief, but, on the other hand, the question might be raised as to how far such restriction could go without seriously hampering the quick and convenient prosecution of business.

Mr. Crosby refers to the arcaded "Rows" of Chester, England. Aside from their quaintness, these arcades are of special interest in this discussion, not because of the traffic relief they afford, but rather because of their popularity for business, notwithstanding that they are in the second story (Fig. 17), as compared with that of frontage on the exterior sidewalk at grade.

Mr. Davison regards the "saturation point" in the use of the pleasure car as impending and urges the controlling of parking and the enforced use of by-passes, while Mr. Seabury stresses the regulation of parking as affording a substantial relief. The writer does not concur in the view that the use of motor vehicles has reached its limit except where congestion has become so pronounced that driving is unpleasant, but believes that the relief of the congested districts would be at once followed by an increased use of motor vehicles

* Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

in such areas. As to the relation of parking to the traffic problem in general, there is much confusion of thought. The long continued occupancy of needed roadway space by a parked car cannot be condoned, but, on the other hand, the right to stop a car long enough for passengers to alight would seem to represent the minimum demand. Such stoppages, particularly if they are coupled with those incident to bus service (which the writer believes must be recognized as a coming serious factor in the use of public streets), practically destroy the value of the traffic lane nearest the curb except for by-passing, and then only where the roadway is insufficient for the accommodation of two moving lanes of vehicles of maximum width in addition to those at the curbs. The immediate prohibition of parking would seem to be a step so drastic as to demand caution; parking is essential if the roadways are to continue to make places of business accessible and cities attractive and thus more prosperous.

The platoon system proposed by Mr. Goodrich is an ingenious device to obtain the advantages of a one-way street and yet permit travel in both directions. If car movement could be more closely regulated, this method could probably be adopted with advantage, but any irregularity would cause serious interference with cross traffic and possibly prevent its being successfully maintained.

Mr. Miller contests the claim that in crowded districts the bus and taxicab have less objectionable effects on traffic than the surface car. The stopping of a trolley forces all other vehicles moving in the same direction to stop, whereas with bus service this is not the case, and, besides, the traffic lanes used by trolley cars are shunned by other classes of traffic owing to the continuous interruption of movement. In further support of the writer's views, attention might be called to the recent proposals made by the two largest traction companies in New York for the introduction of bus lines, which requests presumably would not have been made except for the economic advantages of this method of transportation over the trolley. In London, notwithstanding the narrowness of the roadways, busses are considered as more attractive to the rider than the tram. This view is conclusively shown by the exclusion of surface cars from the old city (although they are municipally owned), and also by the fast growing popularity of the bus service elsewhere.

Mr. Grunsky suggests what would appear to be a common-sense procedure for securing adequate street width for anticipated future needs, the excess width over the requirements of the present to be leased back to the original owner on a rental basis sufficient not only to carry the cost of acquiring the excess width, but also partly to amortize it at the end of the rental period. The City of New York within recent years has secured legislation permitting a somewhat similar procedure. The excess width is first designated on the city plan as courtyard space, whereupon title may be taken by the city subject to the condition that the original owner may retain its use for a prescribed period common to all the parcels thus designated along the line of a street. It has been expected that property owners would be compensated for damage on the basis of the present worth of the award, this, in turn, being based on the probable value of the land and buildings at the time the city takes possession. Thus, it was hoped that building damages in particular could be kept within reasonable limits.

The advisability, as well as the legality, of this procedure has been questioned, and its operation has been limited to a few test cases through which it is hoped to obtain conclusive evidence as to how the damages would be appraised by the Courts. In the first of these cases, buildings on the property were actually removed by the owner before the condemnation proceeding was completed in order to give way to an improvement of a more substantial character, and although payment for building damage was avoided, the question at issue was not settled. It is claimed by opponents of the plan that the procedure will cloud the title and thus make the land and buildings less readily salable. Even should the method eventually come into general use, it is evident that it will not solve the needs of the hour where congestion requires immediate relief.

Since the paper was written, the writer has made further investigations into the question of traffic congestion and remedies, including traffic counts in New York, and an examination of the Paris arcades and of traffic conditions in Paris and London. The seriousness of the situation in New York is perhaps best evidenced by the results of timing vehicular movement within an area extending approximately ½ mile in all directions from the intersection of Fifth Avenue and 42d Street during the period of peak-load traffic. These tests show a speed for north and south movement, ranging from a maximum of 4.5 miles per hour on Fifth Avenue to a maximum of 12 miles per hour on First Avenue, averaging about 7.1 miles per hour; and for east and west movement, ranging from 4.75 miles per hour on 42d Street to 6.5 miles per hour on 44th Street, averaging about 5.7 miles per hour. In the writer's judgment, with proper car spacing regulated according to speed and under most favorable conditions, it is not practicable to move more than from 1 800 to 2 000 vehicles per lane per hour. Traffic on the Manhattan Bridge for 5-min. periods on successive occasions has been observed to exceed the rate of 1 400 vehicles per lane per hour without congestion, and if the terminals were adequate the maximum rate suggested could doubtless be reached. On the other hand, cross-traffic interference is responsible for cutting down the capacity in Fifth Avenue to an average of a little over 800 vehicles per hour per lane, and to an observed maximum for 5-min. periods of from 1 000 to 1 100 vehicles per hour per lane.

An extreme case of congestion and excess movement is presented at the viaduct over 42d Street on the line of Park Avenue in New York. Park Avenue has a roadway width of 72 ft.; practically all its traffic passes over the viaduct, which ranges in width from 36 to 40 ft. with two rectangular turns at the corners of the Grand Central Station. Traffic on the viaduct at the periods of maximum intensity moves in four lanes except in rounding the corners referred to where it is obliged to divide into two lanes. At the turn, the rate for 5-min. periods has exceeded 3 000 vehicles per hour for each lane.

Within the last few months the signal system has been extended to include a large number of the north and south arterial streets, parking has been curtailed in the congested sections, and most of the east and west streets south of Central Park have been made one-way streets. It would seem that little more can be accomplished without going beyond the limits of police measures.

In contrast to the annual rate of increase in traffic in New York, heretofore conservatively estimated by the writer as 15%, statistics recently furnished

FIG. 17.—"THE ROWS", DUKE OF WESTMINSTER'S BUILDING, BRIDGE STREET, CHESTER, ENGLAND. CONTINUOUS ARCADE THROUGH SECOND FLOOR.

FIG. 18.—PARIS, FRANCE: RUE DE RIVOLI, OPPOSITE THE TUILERIES, LOOKING EAST.

Fig. 19.—Arcade Through Louvre Department Store, Rue de Rivoli, at Palais Royale, Paris, France. Provision Has Been Made for Sidewalk Exterior to Arcade

Fig. 20.—Paris, France: Rue des Pyramides, Looking Through Arcade from Place de Rivoli.

FIG. 21.—PARIS, FRANCE: LOOKING THROUGH THE RUE DE RIVOLI ARCADE.

FIG. 22.—PARIS, FRANCE: RUE DE RIVOLI, LOOKING EASTWARDLY FROM RUE DE MONDOVI AT WEST END OF ARCADED SECTION.

by the Traffic Department of the Metropolitan Police of London show a much lower rate, reaching only about 10% at Hyde Park corners (the Hyde Park end of Piccadilly, the point of maximum congestion). On the other hand, in old London, there is little yearly change, a condition due in part to the lack of street capacity and in part to the absence of the skyscraper. Traffic congestion there is caused largely by the busses, the percentage of private cars being small in comparison with the United States and averaging for England only about 1 for every 40 persons. Traffic regulation has not begun to reach the stage obtaining in New York and is generally limited to the policing of all important intersections. The traffic officers are not assigned to a permanent post, nor are they retained continuously in this character of service as is the case here; although they are very efficient, it is manifest that under this system they cannot become as expert as the New York police. One-way traffic has long been in force during certain hours at congested points, and it is now being extended. The speed is limited to 20 miles per hour, except in congested areas, where 10 miles per hour is the limit. At present, the situation is considered sufficiently serious to warrant an investigation as to methods of bringing relief. Various plans are being offered for supplementing the few wide streets of London and attention is being directed to the comprehensive investigation made by the Royal Commission on London Traffic in 1905, which proposed a large number of projects designed to meet the needs of the congested parts of the city. The work done at that time evidently failed to receive support on the ground that it was too visionary.

In Paris, the narrower streets of earlier days have been relieved by a comprehensive system of boulevards. With a few exceptions, the traffic, which consists largely of taxicabs, flows freely through the city with little police supervision and apparently in the absence of a speed limit. In some of the narrower streets, one-way regulations are in force.

Paris probably affords the best example of securing additional street capacity through the introduction of sidewalk arcades, reference to which has already been made by the writer. Rue de Rivoli forms the eastwardly extension of Champs Elysées, one of the most attractive and capacious boulevards of the world; and, although it is flanked on each side with boulevards, it has a strategic position such as to make it one of the most important streets of the city. The exclusive character of the shops that line this street in the arcaded section, comprising a length of about ¾ mile, should remove the fears frequently expressed as to the depreciation of values due to the arcades, particularly when contrasted with those in the unarcaded section. Similar remarks as to values apply to Rue Castiglione and Rue des Pyramides, where the sidewalks on both sides are placed in arcades. (See Figs. 18, 19, 20, 21, and 22.)

As the skyscraper areas grow, it is quite clear that the acquisition of property for new streets becomes increasingly impractical and also that each new building increases the traffic load. The writer's later investigations lead him more strongly than ever to the conclusion that for such areas the ultimate remedy will be found in arcading the sidewalks and in adding the present sidewalk space to the roadways.

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Paper No. 1557

RESEARCHES ON THE STRUCTURAL DESIGN OF HIGHWAYS BY THE UNITED STATES BUREAU OF PUBLIC ROADS*

By A. T. Goldbeck,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Prévost Hubbard, Jacob Feld, and A. S. Woodle, Jr.

Synopsis

The United States Bureau of Public Roads, in connection with its extensive work on Federal Aid Roads, has been authorized to do considerable research involving various phases of highway design and construction. This work has been carried on in different localities and with several co-operative agencies, but largely from the main office and at the Experimental Farm in Arlington, Va. This paper treats only those phases of the research concerned with the structural features of highway design, considered under (1) researches on the sub-grade; (2) loads on pavements; (3) stresses in concrete pavements; (4) Pittsburg, Calif., test road; (5) studies in materials; and (6) miscellaneous investigations. In all cases, the scope and procedure of the test project is given, results are described, and conclusions drawn. Although certain phases have yielded conclusive results, the work on highway research, as a whole, is far from complete; this paper simply aims to acquaint engineers with some of the accomplishments to date and to stimulate further development.

Introduction

It has been estimated that the Federal Aid Highway System when completed, will comprise at least 180 000 miles of road; this will be less than 7%

* Presented at the meeting of the Highway Division, New York, N. Y., January 17, 1924.
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of the road mileage of the United States. At the present time (1924), approximately 45 000 miles of the Federal Aid System are completed or under construction so that the uncompleted mileage of roads necessary to serve adequately present and future traffic reaches astounding figures. Construction of such magnitude requires the expenditure of millions of dollars and the complete investigation of both economic and technical features. Not all of the construction will be high-class pavements; much will be of a secondary nature carrying so little traffic that large expenditures on them would be economically unjustifiable. The main routes, however, and many secondary routes will carry a sufficient volume of travel to warrant paving them with the highest types of surfaces.

The proper adjustment of the road type to local conditions is, of course, a problem involving many considerations for, after all, the road is merely one of the elements forming a transportation system which should function as a whole with greatest economy. Full recognition is made, therefore, of the fact that the final design and choice of road type are governed by factors that must be given proper weight when considered in combination with another important factor, the ability of the roadway to carry traffic loads. The present discussion includes only those investigations which have been undertaken to shed light on the principles involved in the structural design of roads.

Fundamentals Involved in the Structural Design of Roads

A roadway considered from the structural standpoint includes not only the surfacing material, but the underlying sub-grade, and also the shoulders and drainage structures. A wide variety of surfacings is used; some are of a more or less rigid and elastic nature, such as concrete; others have less rigidity and are at times quite plastic, such as the bituminous types; still others have practically no resistance to bending, but are highly resistant to heavy loads when given proper sub-grade support, such as the macadam, gravel, and sand-clay types of pavements.

The design of any engineering structure involves: (1) Correct evaluation of the forces or agents which that structure must resist; and (2) the correct proportioning of that structure to resist those forces economically. Due recognition, of course, must be given to the choice of such materials as will be sufficiently resistant under the attacks of the chemical or physical agents. It is difficult to conceive of any engineering structure that is called on to withstand a more complex set of influences than a road surfacing.

The different types of pavements are constructed over sub-grades ranging from swamp lands and plastic clays to hardpan, compacted gravel, and even solid rock. Obviously, the influence of the sub-grade on pavement design has a very important place in highway research. Then, too, the presence of moisture in these different soils is continuously changing their characteristics, the degree of change depending on the nature of the soil. Again, the moisture content affects the volume in some cases as much as 50% of the dry contents. The influence of this phenomenon is apparent when it is considered that vertical motion of the road surface must accompany these changes in volume and that this motion varies between the sides and the center of the road. Frost

action in the sub-grade also has a decided influence on the road surface in producing non-uniform vertical motion and resulting in deterioration of improperly designed pavements.

The effect of natural forces on the road surface itself must also be considered, as, for illustration, (*a*) the humping or "blowing up" of rigid pavements due to the expanding action of temperature and moisture; (*b*) the action of frost and low temperatures in producing horizontal movement, brittleness, or scaling of the different surfaces; (*c*) the disintegrating or softening action of moisture on certain types of materials; and (*d*) the variety of forces exerted by traffic, sometimes static but generally dynamic.

The element of fatigue of certain types of road surfacings enters still further to complicate the problem, for the loads on pavements are repeated millions of times during their life. It might seem to be almost impossible to arrive at a rational basis for road design when all these disturbing factors are known to exist, and possibly an exact application of any theory may be rendered very difficult. Nevertheless, it is felt that a long step forward has been made in the direction of more correct knowledge. The results lend encouragement to the belief that the future design of roads will be placed on a much more rational and scientific basis than it is at present.

Researches on the Sub-Grade

The character of support offered is always a most important matter in connection with the design of any structure, but most emphatically so in the case of a road surfacing. There are a number of ways in which the type of sub-grade material affects the ability of a road surface properly to support the traffic. The very finely divided plastic soils in the presence of moisture may absorb a high percentage of water by capillarity. They then become extremely soft and are easily deformed under the pressure.

The action of frost in causing heaving, due to the swelling of the sub-grade, is likewise well known. The amount of movement is influenced by the type of soil, because some soils are capable of retaining high percentages of moisture and further because the amount of moisture capable of being frozen, in different types of soil, differs widely.

The shrinkage of soils is an important characteristic, one directly responsible for serious cracking in concrete roads. It is frequently the case that evaporation of water from the sub-grade takes place more rapidly near the shoulders than at the center. Naturally, there is then more vertical movement due to soil shrinkage at the sides of the road than at the center, and, in some cases, as in the adobe soils of California, the sub-grade shrinks entirely away from the road slab so that a serious longitudinal crack develops. Such illustrations will suffice to demonstrate the important part which the sub-grade material must play in the successful design of a road surfacing.

Physical Tests of Sub-Grade Materials.—Most road engineers can recognize immediately the very worst types of soils and also the very best types; it is not always possible to distinguish good from bad soils when they lie somewhere in an intermediate range. One important investigation, then, has

consisted in the development of tests that will distinguish the various soil types.

During the past several years, the United States Bureau of Public Roads has been working on the standardization of tests to distinguish and define the various properties of sub-grade materials. These tests may be listed as:

1.—Mechanical analysis.
2.—Water-holding capacity.
3.—Moisture equivalent.
4.—Volumetric change.
5.—Vertical capillarity.
6.—Comparative bearing value.
7.—Slaking value.
8.—Dye adsorption.

Mechanical Analysis.—This test consists of separating the sample by sieving and washing, into sand, silt, clay, and suspension-clay. Sand is that part of the sample which passes a 10-mesh sieve and is retained on a 200-mesh sieve, but which settles out of a mixture of soil and water, after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid.

Silt is that part of a soil sample which passes a 200-mesh sieve and settles out of a mixture of soil and water, after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid.

Clay is that part of the soil sample which remains in suspension after 8 min. subsidence, to a depth of 8 cm. from the surface of the liquid, but which is thrown down when a centrifugal force equal to 500 times the force of gravity is exerted on the suspended material for a period of ½ hour.

Suspension-clay is that part of the soil sample which remains in suspension when a centrifugal force equal to approximately 500 times the force of gravity is exerted on the suspended material for a period of ½ hour. This is the very finest ingredient of the soil sample.

Water-Holding Capacity.—The water-holding capacity of the soil is the maximum percentage of water which it is capable of retaining.

Moisture Equivalent.—The moisture equivalent of the soil is the percentage of moisture which is retained by the soil after it has first been saturated and then subjected to a centrifugal force equal to 1 000 times the force of gravity for a period of 1 hour. This seems to be a very important test and is a measure of the difficulty of drainage.

Vertical Capillarity.—In this test, the percentage of moisture taken up in a given quantity of the soil by vertical capillarity is determined.

Comparative Bearing Value Test.—In this test, the load-supporting value of the soil is determined.

Volumetric Change.—This test is one to determine the amount of shrinkage that takes place as the soil changes from a condition of definite moisture to an air-dried condition.

Slaking Value.—In this test is determined the length of time required for a briquette of definite size to fall to pieces when immersed in water.

Dye Adsorption Test.—Different soils have the ability in greater or lesser degrees of decolorizing dye solutions which are filtered through them. The variability of dye adsorption properties by soils seems to be correlated with variability of other properties.

A complete description of the details of these tests has been published elsewhere.*

Discussion of Physical Tests on Soils.—It cannot yet be said that the results of the physical tests have been sufficiently correlated with the field behavior of sub-grade materials to establish the limits which define good or poor sub-grade materials.

Mechanical Analysis.—In general, it is quite evident from field observation that soils having a high percentage of the coarse particles are those which give the best results. Such soils, in general, have very low moisture capacity, low capillary absorption, low volumetric change, due to moisture changes, and comparatively high bearing value, no matter what their moisture content may be. On the other hand, soils having high percentages of the very finely divided materials, such as suspension-clay and clay, are the so-called "heavy" soils, which, in general, have high volumetric change, high capacity for moisture, and a high degree of plasticity when they become moist, even with capillary moisture. It remains to be established, however, what the safe limits of suspension-clay and clay might be in a soil for use under the road surface.

Water-Holding Capacity.—The water-holding capacity of a soil is related to the mechanical analysis, since soils with a high percentage of very fine particles, usually have high capacity for holding water. High water-holding capacity in a soil, therefore, is indicative of a poor sub-grade material for such soils are hard to drain and, moreover, generally, but not always, swell and shrink a great deal with changes in moisture. Because of their high moisture content, frost action is apt to cause considerable heaving of the overlying road surface.

Moisture Equivalent.—The moisture equivalent seems to be closely related to the mechanical grading and is apparently a measure of the difficulty of drainage. At the same time, it seems to be a test which gives promise of becoming almost a single measure of the sub-grade-making properties of a soil. Soils having high moisture equivalent are difficult to drain, have high moisture-holding capacity, are apt to be plastic when wet, to swell and shrink a great deal, and will probably heave unduly under frost action. At present, it is considered that moisture equivalent is perhaps the most significant test for sub-grade materials of all those described. There are some field investigations in the West which have indicated that a moisture equivalent of twenty may be taken, at least tentatively, as a rough dividing line between desirable and undesirable sub-grades.

Volumetric Change.—This is an important test for soils as it gives a direct indication of the vertical motion (and, unfortunately, non-uniform vertical motion), which is apt to accompany changes in moisture content of the sub-grade.

* "Physical Properties of Sub-Grade Materials", by J. R. Boyd, *Proceedings*, Am. Soc. for Testing Materials, Vol. 22 (1922).

Vertical Capillarity.—Vertical capillarity is significant in connection with the height of the water-table under the road surface. If a soil has high vertical capillarity, it will absorb a considerable quantity of water from a free source of supply. High vertical capillarity in a sub-grade material is undesirable.

Comparative Bearing Value.—As it is a function of the sub-grade to support the road surfacing with comparatively little deformation, it is necessary to have relatively high bearing value in a sub-grade material. This question is one of considerable complexity and will be discussed later.

Slaking Value.—The slaking value of soils seems to be a measure of the speed with which the various materials will soften under the influence of water. It is not known yet, however, just how to interpret laboratory results in terms of field behavior.

Dye Adsorption.—Soils having a high percentage of colloidal material are capable of decolorizing dye solutions; the dye adsorption test, therefore, furnishes another means for determining to some degree the relative plasticity of soils. In general, soils showing high dye adsorption will have a high percentage of volume change and, in general, are likely to be undesirable sub-grade materials. Tests show that the dye adsorption of the clay and suspension-clay sections of all soils is quite high, but differs considerably among the various parts. The clay and suspension-clay make for lack of stability.

It is quite probable that, in examining a sub-grade soil in the laboratory, all the previously described tests need not be used. It may be found that a sufficiently complete analysis results from the use of a few of these tests. Thus far, the moisture-equivalent test seems to have outstanding value, although it is probable that this test alone is not entirely sufficient. It seems to offer, however, a most direct means for identifying, tentatively at least, poor and good sub-grade soils. Further, it may be performed in the field in such manner as to give almost identical results with the more exact laboratory procedure.

Field Method for Determining Moisture Equivalent of Soils.—This test seems to have been originated by A. C. Rose, M. Am. Soc. C. E., of the U. S. Bureau of Public Roads. The procedure consists of drying a 500-gramme sample of soil at a temperature lower than 100° cent., breaking up the lumps, placing the sample in a bowl, and slowly introducing water from a burette, mixing the water and soil thoroughly together to the consistency of putty. Water is then allowed to drop on the smoothed off surface of the sample and spread with a spoon or spatula. It will be found that before the moisture-equivalent value is reached, the wet, shiny appearance will disappear from the soil surface almost immediately after the drop of water is smoothed over the surface; after the critical value is reached, the surface will retain its wet, shiny appearance. The sample is then dried out and the percentage of water is calculated on the basis of the dry weight. After 0.8% of water is added to this value, the result will be the moisture equivalent of the soil. Tentatively, it can be stated that the approximate dividing line between bad sub-grade soils and good sub-grade soils lies in the moisture-equivalent range of 15 to 20.

Influence of the Bearing Value of the Sub-Grade.—The intensity of pressure on a road sub-grade is governed by a number of factors. The wheel load

is carried directly by the wearing surface which, in turn, transmits the pressure to the underlying sub-grade. If the wearing surface is of a rigid type, as concrete, capable of resisting bending, the sub-grade pressures are distributed over a comparatively broad area adjacent to the wheel, and the highest intensity of pressure is comparatively low. On the other hand, if the surfacing material is of a flexible type, incapable of resisting bending, the area of distribution of sub-grade pressures will be quite restricted and the intensity of pressure under the wheel will be high. These statements are illustrated by curves of pressure distribution obtained under concrete and with broken-stone surfaces of equal thickness, both subjected to a concentrated load. (Fig. 1.) The pressures were measured by soil pressure cells, described elsewhere.*

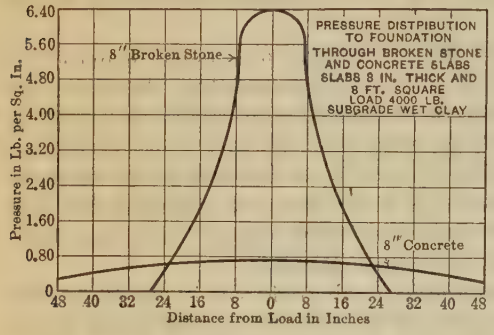
PRESSURE DISTRIBUTION TO FOUNDATION THROUGH BROKEN STONE AND CONCRETE SLABS SLABS 8 IN. THICK AND 8 FT. SQUARE LOAD 4000 LB. SUBGRADE WET CLAY
8" Broken Stone
8" Concrete
Pressure in Lb. per Sq. In.
6.40
5.60
4.80
4.00
3.20
2.40
1.60
0.80
0
48 40 32 24 16 8 0 8 16 24 32 40 48
Distance from Load in Inches

FIG. 1.

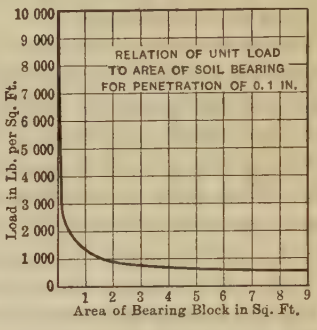

FIG. 2.

The degree of yielding of the soil also affects the pressure distribution. Thus, a solid rock sub-grade would have high and restricted pressures under the wheel load, whereas a soft soil would have low pressure intensities, well spread out, provided, of course, the over-lying slab was of such a nature as to resist bending. The flexibility of the slab likewise controls the pressure intensity and distribution. Rigid slabs, such as concrete, can deflect little before a crack is produced; other types of pavements of less rigid character can be deformed more, without the accompanying incipient failure. The sub-grade material, therefore, has to develop its resistance to the over-lying load within safe limits of deformation of the pavement. It is quite important that the bearing value be obtained corresponding to a known amount of deformation of the sub-grade. This amount will be less with rigid slabs than with non-rigid slabs.

Influence of Bearing Area on Bearing Value of Soils.—Another consideration in determining the bearing value of sub-grades is that due to the different areas of support offered by the sub-grade, depending on the rigidity of the road slab. The bearing value of a soil, for a given penetration of the bearing area, depends on the magnitude of that area. It was with these considerations in mind that a series of tests were conducted to determine the influence of the size of areas on the bearing value of soils for definite penetrations of those bearing areas into the soil. These investigations were conducted using bearing

* "The Distribution of Pressure Through Earth Fills", by A. T. Goldbeck, *Proceedings*, Am. Soc. for Testing Materials, 1917.

blocks of from several square inches up to 9 sq. ft. in area, and with different types of sub-grade materials, such as plastic clay and sand. Although there is some variation in the law for the different kinds of materials, the accompanying curve, Fig. 2, is, in general, indicative of results obtained. It will be noted that when small bearing areas are used, the intensity of pressure required to produce a penetration of 0.1 in. far exceeds that for large sized blocks. This is readily explained by the fact that when the large bearing areas are used a greater thickness of the soil is compressed, which contributes more toward the movement of the block than in the case of the small bearing area. It may not always be possible to make bearing value determinations by the use of large size tests. If smaller bearing blocks are used, it will be necessary to obtain the bearing value of the soil at a much less penetration, in order to obtain values corresponding to a definite penetration of the large bearing block. The relation between the depths of penetration of different sized bearing blocks for like intensities of pressure has been established by a series of tests using various bearing areas.* In Fig. 3 is shown the results of this investigation, giving the relative penetrations of the areas under equal intensities of load. The experimental results for a penetration of 0.1 in. over the area of 9 sq. ft. seem to be quite accurately expressed by the relation:

$$\text{Area (square feet)} = 900 \times [\text{penetration (inches)}]^2$$

From this equation, it follows that:

$$\frac{\text{Penetration } (a)}{\text{Penetration } (A)} = \sqrt{\frac{a}{A}}$$

in which, a and A are the areas of the respective bearing blocks. Other experimenters have confirmed this relation.†

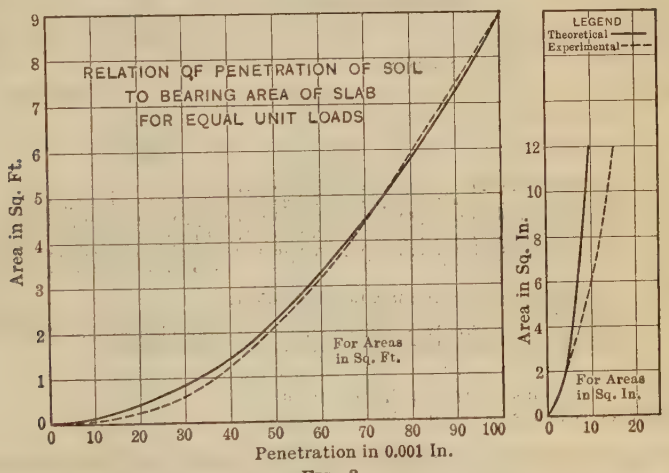

FIG. 3.

Experiments in Sub-Grade Moisture.—Even the worst kind of sub-grade material has a high bearing value when it is dry. On the other hand, the

* The details of this investigation have not yet been published.

† See article in *Le Genie Civil*, May 26, 1923, by Bijls, of Belgium.

best kind of sub-grade materials are in their best conditions when slightly moist; this applies especially to clean sand. The worst sub-grade materials are dangerous only when they have taken up a sufficient quantity of moisture to cause them to swell or to render them plastic. It is important, therefore, that complete information be obtained as to the laws governing moisture in soils.

Drainage Experiments.—In an effort to determine the effectiveness of a number of different systems of drainage, slabs, 14 ft. sq., were laid as shown in the cross-sections of Fig. 4. It will be noted that the sub-grades under these slabs were tile-drained in some cases, that all had deep side ditches, and that several had deep impervious walls surrounding the slabs. Each slab was perforated with holes through which samples of the underlying sub-grade could be obtained. The accompanying data (Table 1) gives the results of these moisture determinations for each of the various sections.

As a result of these experiments, it is concluded that systems of drainage merely remove the free water in the sub-grade, but cannot remove capillary moisture, and it seems that notwithstanding the different systems of drainage used, there were high moisture contents under all the slabs at some time during the year. The impervious cut-off walls built along the sides did not serve particularly well to reduce the moisture content under the slabs. Admixtures of water-gas tar were likewise not particularly effective. It must not be inferred from these statements that tile-drains and ditch drainage are not considered useful, for such methods remove free water, although they do not remove capillary water.

TABLE 1.—TABLE OF MOISTURE VALUES IN TEN TREATED SUB-GRADES (FIG. 4), AT ARLINGTON, VA. AVERAGE MOISTURE CONTENT OF FIRST INCH OF DEPTH.

Date.	Section.										Remarks.
	1	2	3	4	5	6	7	8	9	10	
1921:											
March 2	20.2	18.9	19.8	20.9	16.8	9.7	19.7	17.6	20.5	17.2	
April 18	20.4	16.2	18.8	16.8	16.7	16.3	18.6	17.6	20.1	18.8	Sprinkling began April 8.
May 6	18.4	19.5	21.0	19.2	19.8	19.6	19.4	18.3	18.8	17.9	Sprinkling stopped May 27.
August 5	19.2	20.0	19.1	20.4	15.3	14.2	20.8	19.8	19.8	17.1	Ditches filled with water July 14.
November 7	16.2	19.1	20.3	17.9	19.2	16.8	17.1	17.2	16.4	20.6	Ditches drained and cleaned. Ground frozen November 17.
1922:											
January 25	37.3	20.7	24.1	24.6	21.6	18.2	19.2	21.0	24.5	21.4	
February 20	24.9	22.3	21.8	19.3	14.6	14.7	18.5	20.8	21.2	15.8	Ditches filled with earth March 27.
May 23	22.1	17.6	21.1	18.7*	12.4	5.9	19.8	16.8	18.2	17.2	Sprinkling began June 12.
July 17	22.7	23.4	27.0	25.2†	15.8	17.4	25.6	22.5	24.5	22.7	Heavy rains.

* Average of readings at ¼ in., ¾, and 1 in.
† Average of readings at ¼ in., ½ in., and ¾ in.

Curative Treatments for Sub-Grades.—If a road is to be built in a location where poor sub-grade material exists, what effect will the sub-grade have on the design of the road surface? Is it possible to convert a plastic type of sub-grade into a more desirable non-plastic type through some system of curative treatment? Experiments to determine these points have been conducted using various admixtures to soil, with the results shown in Fig. 5,

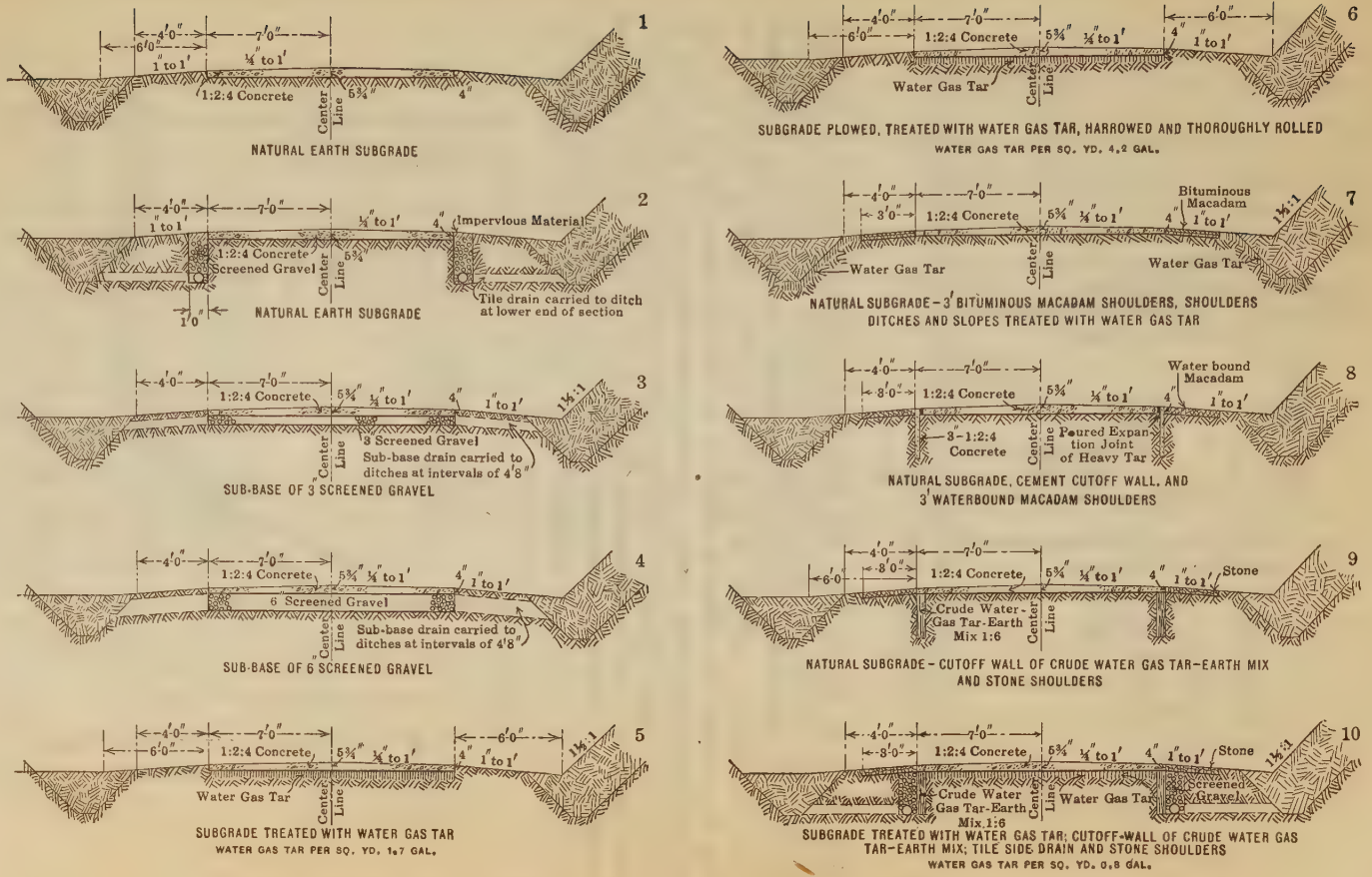

FIG. 4.—CROSS-SECTIONS, PAVEMENT TYPES 1 TO 10, FOR DRAINAGE AND SUB-GRADE EXPERIMENTS.

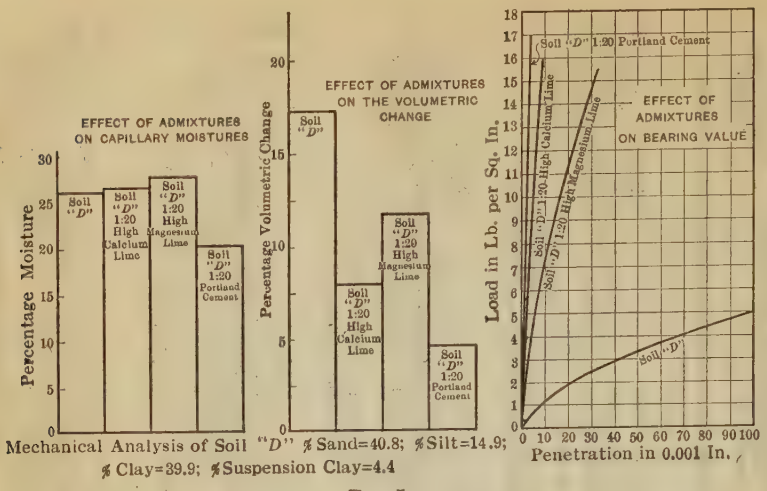
EFFECT OF ADMIXTURES ON CAPILLARY MOISTURES
Percentage Moisture
Soil "D"
Soil "D" 1:20 High Calcium Lime
Soil "D" 1:20 High Magnesium Lime
Soil "D" 1:20 Portland Cement
EFFECT OF ADMIXTURES ON THE VOLUMETRIC CHANGE
Percentage Volumetric Change
EFFECT OF ADMIXTURES ON BEARING VALUE
Load in Lb. per Sq. In.
Soil "D" 1:20 Portland Cement
Soil "D" 1:20 High Calcium Lime
Soil "D" 1:20 High Magnesium Lime
Soil "D"
Penetration in 0.001 In.
Mechanical Analysis of Soil "D" % Sand=40.8; % Silt=14.9; % Clay=39.9; % Suspension Clay=4.4

FIG. 5.

from which it appears that 5% additions of hydrated lime and Portland cement have the effect of decreasing the volumetric change due to variations in moisture content and likewise of increasing the bearing value of plastic soils that contain moisture up to a limit of their capillary moisture. In Fig. 6 is shown the effect of additions of different quantities of sand for accomplishing the same result. It would seem then that it is possible to create over a bad sub-grade a layer which has high bearing value, low moisture adsorption, and low volumetric change, by the expedient of mixing and harrowing into the soil admixtures such as lime, Portland cement, or sand, or by the use of a porous, granular layer. Some experiments made by the

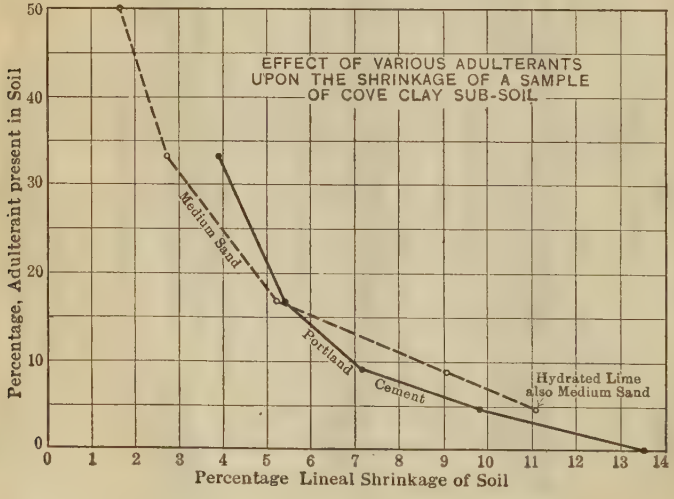
EFFECT OF VARIOUS ADULTERANTS UPON THE SHRINKAGE OF A SAMPLE OF COVE CLAY SUB-SOIL
Percentage, Adulterant present in Soil
Medium Sand
Portland Cement
Hydrated Lime also Medium Sand
Percentage Lineal Shrinkage of Soil

FIG. 6.

U. S. Bureau of Public Roads a number of years ago illustrate why it is beneficial to use a layer of material of high supporting value interposed between the plastic sub-grade and the overlying road slab. The resulting curves (Fig. 7) show the distribution of pressure intensity due to concentrated loads placed on top of layers of different thicknesses. As would be expected, the thicker the layer, the smaller is the intensity of pressure at the bottom and the wider is that pressure spread. Hence, a layer of material of good supporting value has the effect of decreasing the intensity of pressure on the soft sub-grade due to concentrated loads on the road surface. Under just what conditions it is economical to use such a layer, created either by admixing with the sub-grade, or by placing on top of the sub-grade, is yet to be demonstrated.

Loads on Pavements

It is quite necessary to know not only the static weights of the motor vehicles operating over the highways, but also the impact pressures produced on pavements by trucks moving at high speeds and equipped with a wide variety of tires. In a large number of tests of this nature, an effort has been made to obtain the relative impacts under the various conditions that might exist due to road roughness. In Fig. 8 is illustrated the motion both of the body of the truck and of the unsprung portion when the rear wheels fall from a definite height and strike the road surface.

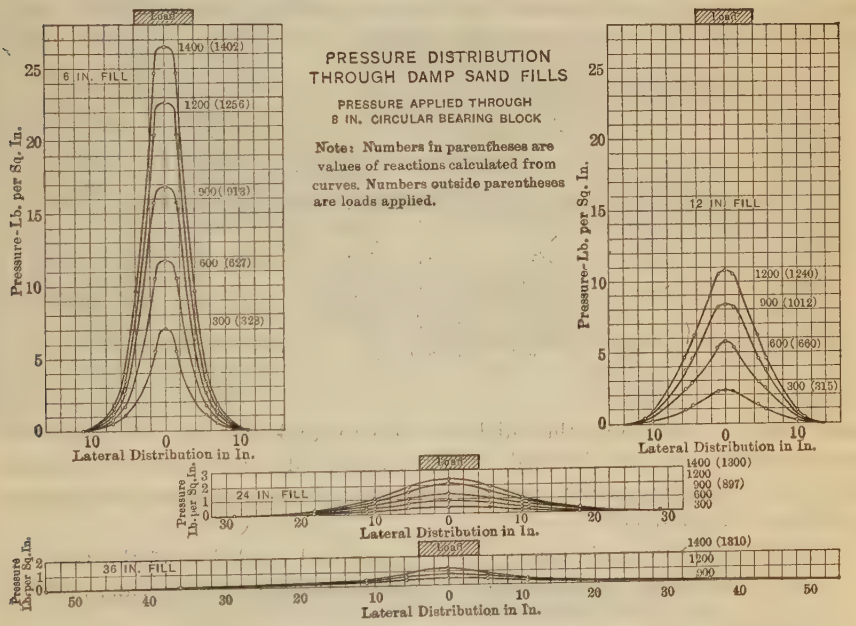

Fig. 7.

Ordinarily, the springs of the truck are under static compression, due to the body load so that the instant compression is relieved, owing to a depression in the road surface, the wheels are shot downward under the

force of gravity plus the pressure of the springs; but the body of the truck descends much more slowly. At the instant the tires come into contact with the road surface again, upward pressure on the wheels begins and downward vertical motion of the wheels is finally reduced to zero. The pressure on the road surface has increased from zero at the instant of initial contact with that surface, to a maximum when the tire is fully compressed. To obtain a measure of this pressure, the following method of test was used.

Cylinders were prepared in large numbers by cutting ½-in. lengths from copper bars ½ in. in diameter. All the cylinders were thoroughly annealed under the same conditions, in order to render them equally resistant to loads. A hydraulic jack was set on a concrete base and the plunger, provided with a platform, was set at about the same level as the road surface, one of the annealed copper cylinders being then placed under the plunger of the jack. The motor truck was driven so that its rear wheels came into contact with this plunger, and the full force of the blow was delivered to the copper cylinder, thus permanently deforming it. The pressures corresponding to various cylinder shortenings having been determined separately, the effect of the blow was readily known in terms of these deformations. In some cases, the truck wheels mounted a slight incline and dropped from definite heights to the plunger; in other cases, obstructions were placed on the plunger. A large variety of truck tires were used on the same truck, with like conditions of loading, and a number of different kinds of trucks, operated over a wide range of speed. In this manner, the influence of the weight of the truck, the sprung and the unsprung weight, the speed, kind of tire—whether pneu-

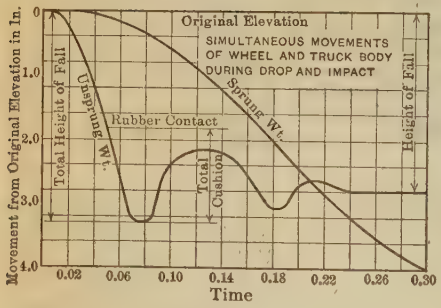

FIG. 8.

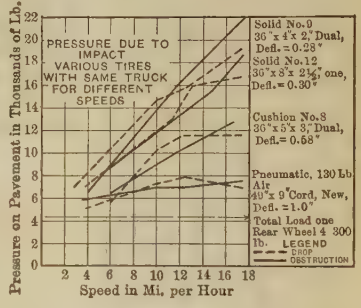

FIG. 9.

matic, cushion, or solid,—the thickness of tire, and the type of inequality in the road surface, was obtained. Typical results of these tests are indicated in the accompanying curves* and the following conclusions have been drawn (see Figs. 9, 10, and 11):

(1) Impact depends largely on the kind and condition of the tire.

(2) Thin or worn solid rubber tires, even though they be wide, produce high impact forces.

(3) Pneumatic tires offer the greatest aid in reducing impact forces; with their use, the impact increases only slightly with the speed of the truck.

* See, also, *Public Roads*, March and December, 1921.

FIG. 10.—IMPACT TEST, TRUCK PASSING OVER MEASURING APPARATUS.

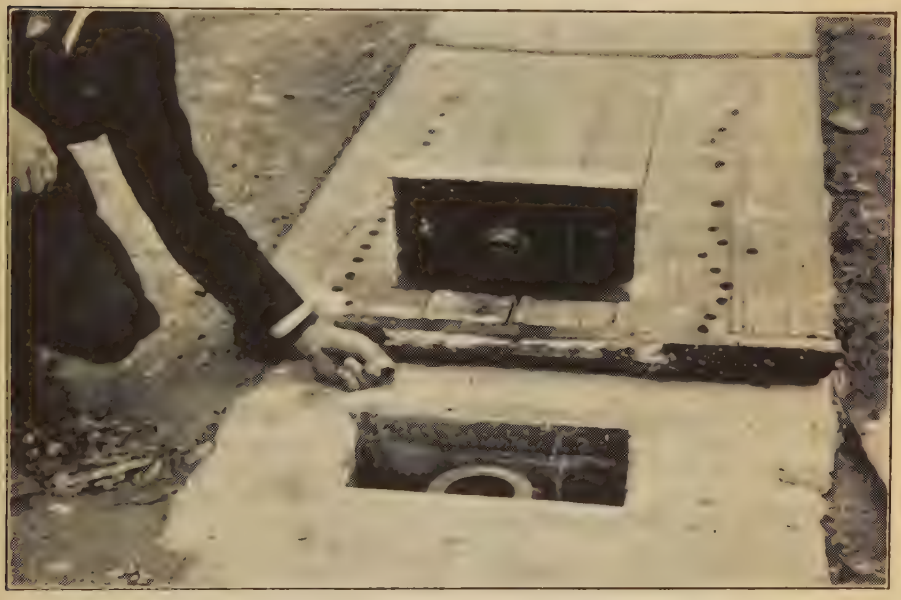

FIG. 11.—DETAILS OF COPPER CYLINDER DEVICE.

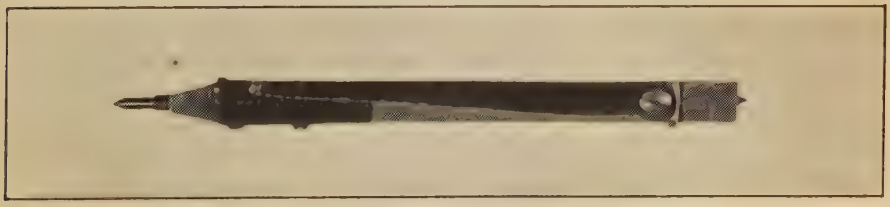

FIG. 12.—GRAPHIC STRAIN-GAUGE.

(4) Cushion tires, that is, tires having a degree of softness and deflection between solid and pneumatic types, offer corresponding advantages in reducing impact.

(5) Impact increases with the speed of the truck, but it cannot be said to increase according to any constant ratio or power of the speed.

(6) Although heavy unsprung weight may give higher impact than lighter unsprung weight, it cannot be said that this is the major controlling factor.

(7) The relative destructive effect produced by light weight, high-speed trucks, and heavy, slow-moving trucks has not been determined by these tests. They do indicate, however, that equal impact may be obtained under some conditions.

(8) Impact may be as high as seven times the static load on one rear wheel when a solid-tired truck strikes a 1-in. obstruction at 16 miles per hour, an average value being about four times. For pneumatic tires, the maximum impact value is probably not more than $1\frac{3}{4}$ times the load on one rear wheel, and an average value is not more than $1\frac{1}{4}$ times the load.

(9) All cushion wheels do not reduce impact on the road surface even though they may cushion the vehicle.

Other devices are now being used in similar investigations with a promise of obtaining successful results. One of these devices is the Kreüger cell* and the other a recording accelerometer for measuring the vertical deceleration of the truck wheels at the instant of contact with the road surface. Knowing the value for deceleration, the maximum pressure due to impact is determined by the formula:

$$F = M\,(a + g) + P$$

in which,

M = the mass of the unsprung portion of the motor trucks;
a = the maximum vertical deceleration;
g = the acceleration of gravity; and
P = the spring pressure at the instant of full contact.

Stresses in Concrete Pavements

Another series of tests leading up to the rational design of concrete roads has been that of the determination of the distribution of maximum fiber deformation in the surface of the road slab due to motor-truck traffic. For this purpose, it was necessary to design a special strain-gauge which would record these maximum deformations. It was necessary that this instrument be of the recording type, small and compact.

The type finally evolved (Fig. 12), called a "graphic strain-gauge", consists of a single lever having a multiplication of approximately 70 to 1, the long arm carrying a needle which scratches a mark on a smoked-glass

* "Method for Measuring and Calculating the Magnitude of Forces with Particular Regard to Impact Forces," by H. Kreüger, *Transactions No. 2*, Eng. Science Academy, Stockholm, Sweden.

plate. This mark is approximately seventy times as long as the movement of the plunger of the instrument, which movement equals the fiber deformation. The glass record is "fixed" in a thin varnish solution and then examined under a microscope to obtain the length of the scratch mark. Very consistent results have been obtained with this instrument under static and dynamic loads, and it is known to have practically no "overthrow" under dynamic loads. When used for stress measurements, a slot is cut in the surface of the pavement, $\frac{3}{4}$ in. square and about $6\frac{1}{2}$ in. long, just large enough to receive the instrument the ends of which bear against brass plates cemented at the ends of the slot. Finally, a thin steel plate is laid on the pavement over the instrument to protect it against possible disturbance from the motor-truck tires. A number of gauges being thus placed at various locations in the surface, the motor truck is run over the pavement a number of times in different positions, and after each run the smoked-glass plate in each instrument is moved forward a slight amount, placing it in position to receive another record. In this manner, a dozen or more records may be inscribed on the same smoked glass.

In considering slab design, the maximum deformations in the concrete are of greatest interest, irrespective of the position of the load which causes those maxima. For the purposes of the present determination, the outer wheels of the trucks were run at different distances from the sides of the road. In a plain concrete slab of 6-in. uniform cross-section, the maximum fiber deformations using a 5-ton truck were as shown in Fig. 13.

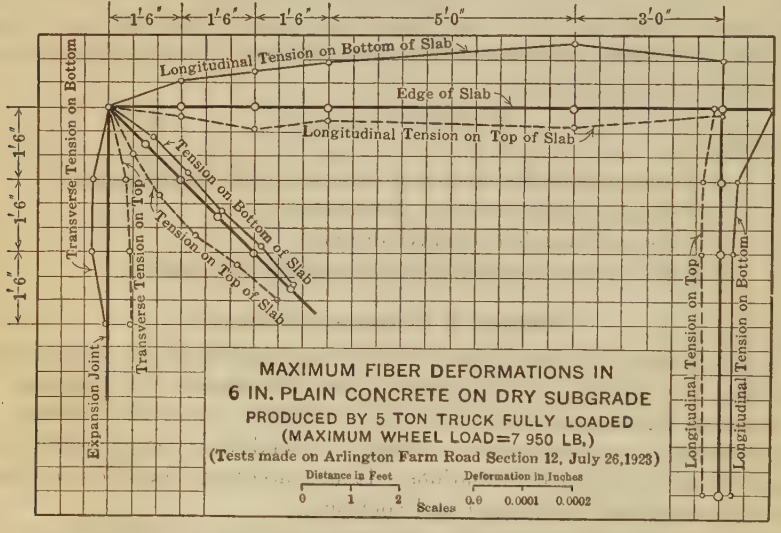

FIG. 13.

It will be noted that there is considerable tension in the bottom of the slab at the edges and also that there is considerable tension on a 45° diagonal in the top of the slab at a corner formed by the intersection of a transverse joint and the edge. The longitudinal stress in the interior of the slab directly

under the wheel load is much less than that at the outer edge, and it would seem certain, therefore, that strengthening of the edge of the slab is necessary to obtain a slab of more nearly balanced design. It is to be noted that the tension in the bottom of the slab along the edge approximates the tension in the top near a corner.

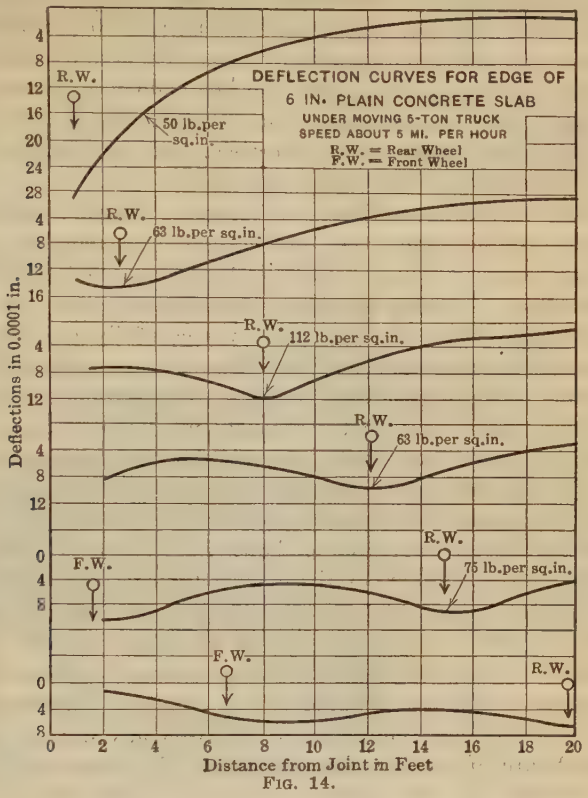

FIG. 14.

Deflection Curves Along the Side of a Concrete Road.—An autographic deflectometer, built specially for the purpose, has been used for obtaining a measure of the deflections along the side of road slabs while taking stress measurements as just described. A view of this instrument is shown in Fig. 15. It consists of a board, 20 ft. long, carrying a series of pen arms actuated by plungers resting on the pavement. The travel of the pen is 100 times the deflection of the pavement and the record is made on a strip of paper moved by an electric motor. As the trucks run past the instrument, each pen arm traces a curve giving an exaggerated graphic representation of the deflection; from the twenty curves thus drawn, the true shape of the elastic curve is obtained. A series of deflection curves, Fig. 14, is shown for a 6-in. slab; similar curves have been obtained for slabs of several different designs. They corroborate the strain-gauge indications that high tension may exist both at the top and bottom of the slab along the edge.

Stress Along Center Joint of a Concrete Road.—The question has often been raised as to whether the stresses due to wheel loads are likely to be as

high along an unsupported center longitudinal joint in a concrete road as along the edge of the slab. Measurements made on a 6-in. concrete road show that these stresses are about equal.

Effectiveness of Dowels in Transmitting Load Across a Center Joint.—If the stress created along a center joint by a wheel load is high, the question arises as to whether dowels may be effectively used to transmit it across to the adjacent slab and thus bring both slabs into action. On viewing the deflection curves of Fig. 14, it is seen that the sudden curvature takes place over a few feet. High stress exists where there is sharp curvature in the elastic line; therefore, dowels, to be effective, must be placed at sufficiently close intervals to eliminate this sharp curvature.

Impact Tests on Pavement Slabs.—In order to obtain a relative measure of the resistance of the many types and designs of pavements to the action of traffic, a large series of impact tests have been conducted at Arlington, Va., in which the action of traffic has been simulated as nearly as practicable.

Before laying the slabs, the sub-grade was carefully prepared so that it would be as uniform as possible. One series of specimens was laid on a sub-grade having side ditches which were kept dry, and a duplicate series with side ditches which were kept constantly filled with water. Tile-drains likewise conveyed water from these ditches into the sub-grade at frequent intervals so that this sub-grade was always in a soft, wet condition. Each of the slabs were 7 ft. square and of a design indicated in Table 2.

The machine used in these tests is shown in Fig. 16. It is a mechanism for producing impacts similar to those of actual motor trucks and has a truck wheel that may be shod with any kind of tire desired, and a truck spring interposed between the frame of the machine and the wheel. Both the frame and the wheel may be loaded to correspond with the sprung and unsprung weights of different sizes of motor vehicles. The actuating mechanism raises the wheel and allows it to fall from any desired height.

In the procedure of making these tests first the machine is mounted in the position desired. In the present series, part of the slabs was tested at the corner and part with the wheel placed at the middle of one edge. An accelerometer was mounted on the unsprung portion of the machine, and the specimens were fitted with graphic strain-gauges of the type already described, also with Ames dials for deflection and autographic means for obtaining vertical movements of both the slab and the wheel. At first, impacts were delivered to simulate a 2-ton truck, beginning with low heights of fall and increasing to a height that produced impacts as high as had been discovered by tests of actual traffic, as determined by the readings of an accelerometer mounted on the rear unsprung portion of the motor truck. When the accelerometer record from the impact machine and that from the motor truck coincided, it was assumed that the forces received by the slab and road surface were equal, inasmuch as the sprung and unsprung weights of the two machines were alike. Auxiliary tests were used to determine the bearing value of the sub-grade under each slab and also the physical characteristics of the concrete. This investigation will not be reported in detail; the results, however, are given in Figs. 18, 19, 20, 21, 22, and 23.

FIG. 15.—VIEW OF INSTRUMENT FOR RECORDING PAVEMENT DEFLECTIONS.

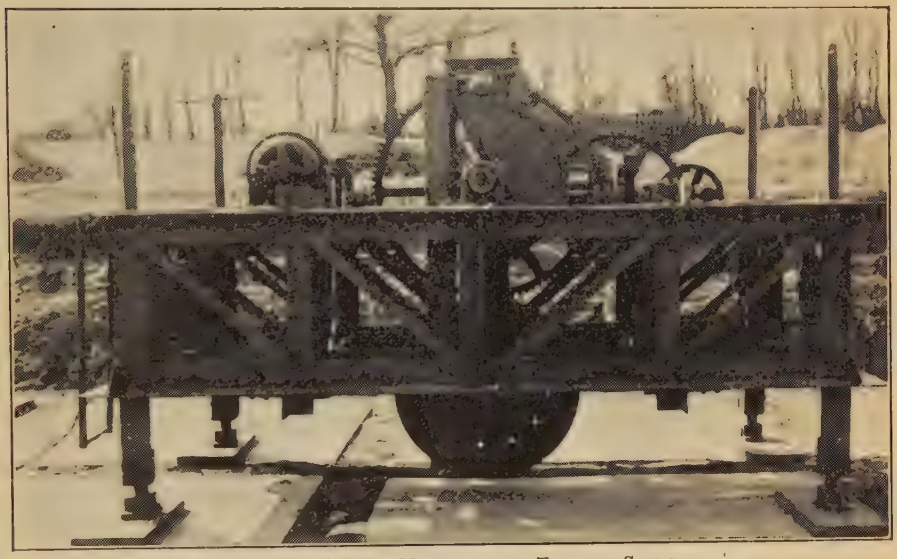

FIG. 16.—IMPACT MACHINE FOR TESTING SLABS.

FIG. 17.—WEAR MACHINE USED AS CONCRETE TRUCK.

TABLE 2.—Slabs for Impact Tests.

(All Slabs are 7 Ft. Square of Material and Thickness, as Shown Below. Series 200-267, on Wet Sub-Grade; Series 300-337, on Dry Sub-Grade; Series 1R-18R, on Dry Sub-Grade.)

Plain Slabs.

Slab no.	Base.		Binder.		Surface.	
	Thickness, in inches.	Material.	Thickness, in inches.	Material.	Thickness, in inches.	Material.
200	6	Macadam	4	Bituminous concrete	2	Topeka
201	6	"	4	" "	2	"
204	4	"	3	" "	2	"
205	4	"	3	" "	2	"
208	4	"	2	.....	2	"
209	4	"	2	.....	2	.....
212	6	"	....	.....	2	Topeka
213	6	"	....	.....	2	"
214	9	"	....	.....	2	"
215	9	"	....	.....	2	"
216	12	"	....	.....	2	"
217	12	"	....	.....	2	"
218	6	Bituminous concrete	....	.....	2	"
219	6	" "	....	.....	2	"
220	4	" "	....	.....	2	"
221	4	" "	....	.....	2	"
222	6	1 : 3 : 6 K*	1½	Bituminous concrete	1½	Asphalt top
223	6	1 : 3 : 6 K	1½	" "	1½	" "
224	6	1 : 3 : 6 K	....	.....	4	C. Bituminous concrete
225	6	1 : 3 : 6 K	....	.....	4	" " "
226	6	1 : 3 : 6 K	....	.....	2	" " "
227	6	1 : 3 : 6 K	....	.....	2	" " "
228	4	1 : 3 : 6 K	....	.....	2	Topeka
229	4	1 : 3 : 6 K	....	.....	2	"
230	4	1 : 1½ : 3 K	....	.....	2	"
231	4	1 : 1½ : 3 K	....	.....	2	"
232	6	1 : 3 : 6 K	....	.....	2	"
233	6	1 : 3 : 6 K	....	.....	2	"
234	6	1 : 1½ : 3 K	....	.....	2	"
235	6	1 : 1½ : 3 K	....	.....	2	"
236	8	1 : 3 : 6 K	....	.....	2	"
237	8	1 : 3 : 6 K	....	.....	2	"
238	8	1 : 1½ : 3 K	....	.....	2	"
239	8	1 : 1½ : 3 K	....	.....	4	1 : 1½ : 3 K
240	..	.....	....	.....	4	1 : 1½ : 3 K
241	..	.....	....	.....	6	1 : 1½ : 3 K
242	..	.....	....	.....	6	1 : 1½ : 3 K
243	..	.....	....	.....	6	1 : 3 : 6 K
244	..	.....	....	.....	6	1 : 3 : 6 K
245	..	.....	....	.....	8	1 : 1½ : 3 K
246	..	.....	....	.....	8	1 : 1½ : 3 K
247	..	.....	....	.....	8	1 : 3 : 6 K
248	..	.....	....	.....	8	1 : 3 : 6 K
249	..	.....	....	.....	2	Topeka
300	4	Macadam	2	Bituminous concrete	2	"
301	4	"	2	" "	2	"
304	4	"	....	.....	2	"
305	4	"	....	.....	2	"
306	6	"	....	.....	2	"
307	6	"	....	.....	2	"
308	9	"	....	.....	2	"
309	9	"	....	.....	2	"
310	6	Bituminous concrete	....	.....	2	"
311	6	" "	....	.....	2	"
312	4	" "	....	.....	2	"
313	4	" "	....	.....	2	"
314	6	1 : 1½ : 3 K	....	.....	2	"
315	6	1 : 1½ : 3 K	....	.....	2	"
316	6	1 : 3 : 6 K	....	.....		

*K = Cement concrete.

TABLE 2.—(*Continued*).

PLAIN SLABS (*Continued*).

Slab no.	Base.		Binder.		Surface.	
	Thickness, in inches.	Material.	Thickness, in inches.	Material.	Thickness, in inches.	Material.
317	6	1 : 3 : 6 K*	....	.....	2	Topeka
318	4	1 : 1½ : 3 K	....	.....	2	"
319	4	1 : 1½ : 3 K	....	.....	2	"
320	8	1 : 1½ : 3 K	....	.....	2	"
321	8	1 : 1½ : 3 K	....	.....	2	"
322	..	.....	....	.....	4	1 : 1½ : 3 K
323	..	.....	....	.....	4	1 : 1½ : 3 K
324	..	.....	....	.....	6	1 : 1½ : 3 K
325	..	.....	....	.....	6	1 : 1½ : 3 K
326	..	.....	....	.....	6	1 : 3 : 6 K
327	..	.....	....	.....	6	1 : 3 : 6 K
328	..	.....	....	.....	8	1 : 1½ : 3 K
329	..	.....	....	.....	8	1 : 1½ : 3 K

SLABS WITH MESH REINFORCING. WET SUB-GRADE.

Slab no.	Surface.			
	Thickness, in inches.	Material.	Reinforcement.	Percentage of steel = p.
250	4	1 : 1½ : 3 K	1 layer No. 6	0.21
251	4	1 : 1½ : 3 K	1 " " 6	0.21
252	4	1 : 1½ : 3 K	2 " " 6	0.42
253	4	1 : 1½ : 3 K	2 " " 6	0.42
254	4	1 : 1½ : 3 K	2 " " 8	0.66
255	4	1 : 1½ : 3 K	2 " " 8	0.66
256	4	1 : 1½ : 3 K	1 " " 10	0.41
257	4	1 : 1½ : 3 K	1 " " 10	0.41
258	4	1 : 1½ : 3 K	2 " " 10	0.82
259	4	1 : 1½ : 3 K	2 " " 10	0.82
260	6	1 : 1½ : 3 K	1 " " 6	0.19
261	6	1 : 1½ : 3 K	1 " " 6	0.19
262	6	1 : 1½ : 3 K	2 " " 6	0.38
263	6	1 : 1½ : 3 K	2 " " 6	0.38
264	6	1 : 1½ : 3 K	2 " " 8	0.46
265	6	1 : 1½ : 3 K	2 " " 8	0.46
266	6	1 : 1½ : 3 K	2 " " 10	0.58
267	6	1 : 1½ : 3 K	2 " " 10	0.58

SLABS WITH MESH REINFORCING. DRY SUB-GRADE.

Slab no.	Surface.			
	Thickness, in inches.	Material.	Reinforcement.	Percentage of steel = p.
330	4	1 : 1½ : 3 K	1 layer No. 6	0.21
331	4	1 : 1½ : 3 K	1 " " 6	0.21
332	4	1 : 1½ : 3 K	1 " " 10	0.41
333	4	1 : 1½ : 3 K	1 " " 10	0.41
334	6	1 : 1½ : 3 K	2 " " 6	0.38
335	6	1 : 1½ : 3 K	2 " " 6	0.38
336	6	1 : 1½ : 3 K	2 " " 8	0.46
337	6	1 : 1½ : 3 K	2 " " 8	0.46

* K = Cement concrete.

TABLE 2.—(*Continued*).

Slab no.	Slabs with Rod Reinforcing. Dry Sub-Grade. Surface. Thickness, in inches.	Material.	Reinforcement.	Percentage of steel = p.
1 R	6	1 : 1½ : 3 K*	1 layer ⅝-in. rods at 3¾ in., cu. cm.	0.5
2 R	6	1 : 1½ : 3 K	1 " " " " " " " "	0.5
3 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
4 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
5 R	6	1 : 1½ : 3 K	2 " " " " " 7⅜ " "	0.5
6 R	6	1 : 1½ : 3 K	2 " " " " " " " "	0.5
7 R	6	1 : 1½ : 3 K	1 " ⅝ " " " 10¼ " "	0.5
8 R	6	1 : 1½ : 3 K	1 " " " " " " " "	0.5
9 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
10 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
11 R	6	1 : 1½ : 3 K	2 " " " " " 20½ " "	0.5
12 R	6	1 : 1½ : 3 K	2 " " " " " " " "	0.5
13 R	6	1 : 1½ : 3 K	1 " ⅞ " " " 20 " "	0.5
14 R	6	1 : 1½ : 3 K	1 " " " " " " " "	0.5
15 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
16 R	6	1 : 1½ : 3 K	2 " " " " " " " "	1.0
17 R	6	1 : 1½ : 3 K	2 " " " " " 40 " "	0.5
18 R	6	1 : 1½ : 3 K	2 " " " " " " " "	0.5

* K = Cement concrete.

It is not intended that these tests shall be used to indicate the actual resistance of the various designs of pavements to traffic loads, but rather the relative value of these various types for carrying traffic under the sub-grade conditions at Arlington. It is not unlikely that, in some cases, these relative resistances might vary considerably as the relation of support offered by the sub-grade and that offered by the slab changes. The tests, however, seem to warrant the following conclusions:

1.—The resistance of a road slab depends in part on the supporting value of the sub-grade. A sub-grade of high supporting value materially increases the resistance to impact.

2.—The resistance of rigid slabs to impact varies neither directly with the depth nor as the square of the depth, but as some power less than two.

3.—In general, plain concrete slabs show no more resistance to impact delivered at the edge than to impact delivered at the corner.

4.—Transverse cracks, and longitudinal cracks near the sides of a road slab, may be caused by impacts delivered at the edge of the slab.

5.—Plain concrete of 1:3:6 mix shows from about 60 to about 80% of the resistance to impact of plain concrete of 1 : 1½ : 3 mix. The lean mix also shows more variation in strength.

6.—Reinforcing steel in concrete slabs, when present in sufficient quantity, and when placed so as to receive tensile stress, adds to the resistance of the slab to impact.

7.—After a crack occurs, small deformed rods closely spaced seem to be more effective than the same percentage of steel in large deformed rods, widely spaced.

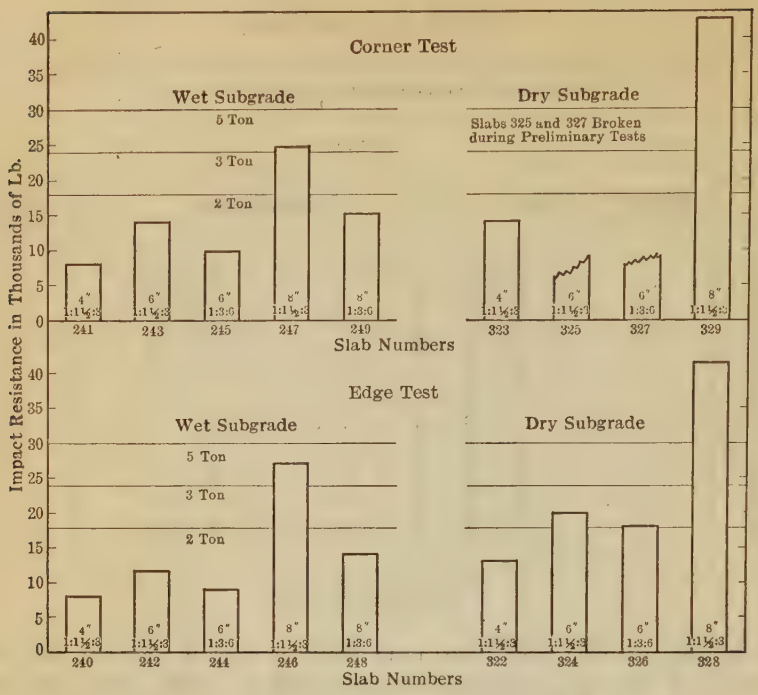

FIG. 18.—RESULTS OF IMPACT TESTS, RELATIVE RESISTANCE OF PLAIN CONCRETE SLABS.

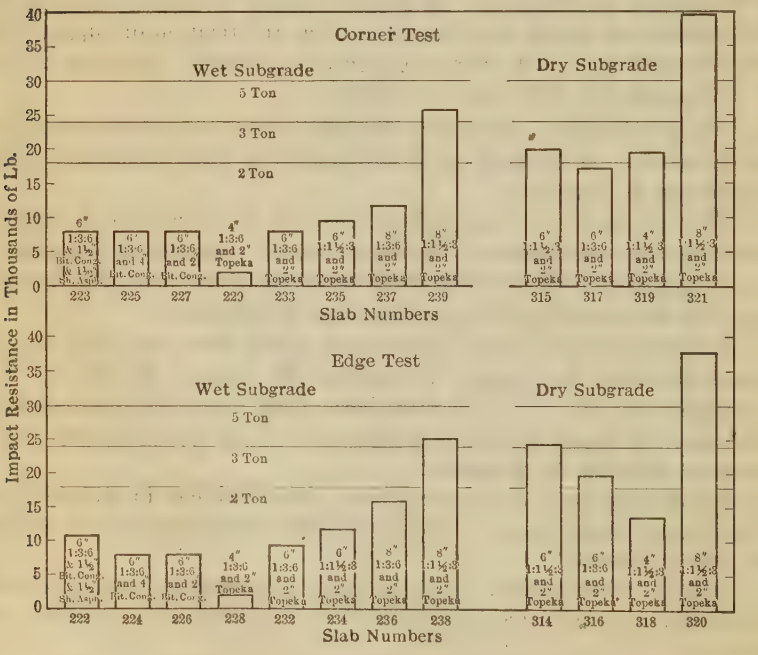

FIG. 19.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF CONCRETE SLABS WITH BITUMINOUS TOPS.

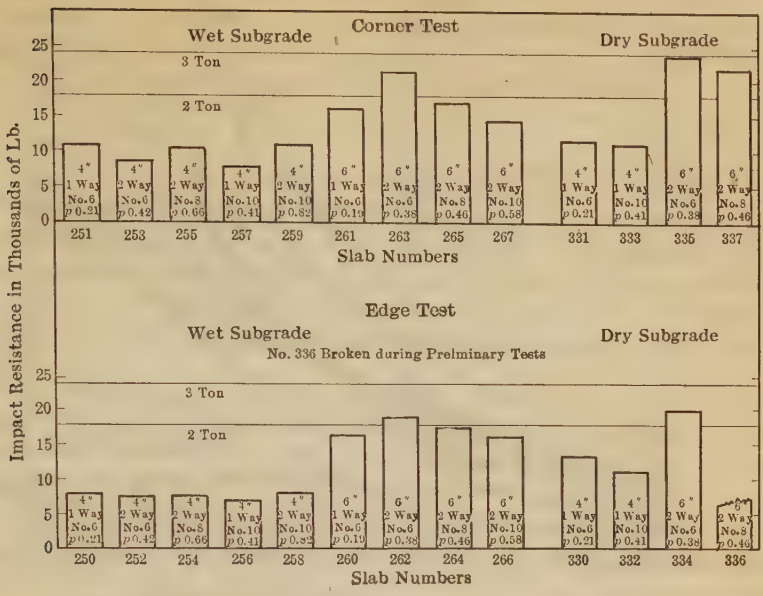

FIG. 20.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF MESH-REINFORCED CONCRETE SLABS, 1 : 1½ : 3 MIX.

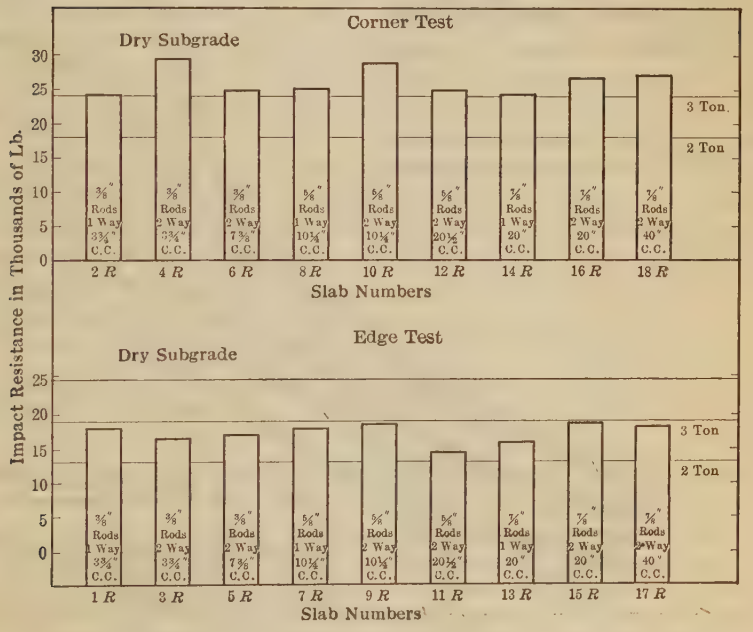

FIG. 21.—RESULTS OF IMPACT TESTS. RELATIVE RESISTANCE OF ROD-REINFORCED CONCRETE SLABS, 6 IN. THICK, 1 : 1½ : 3 MIX.

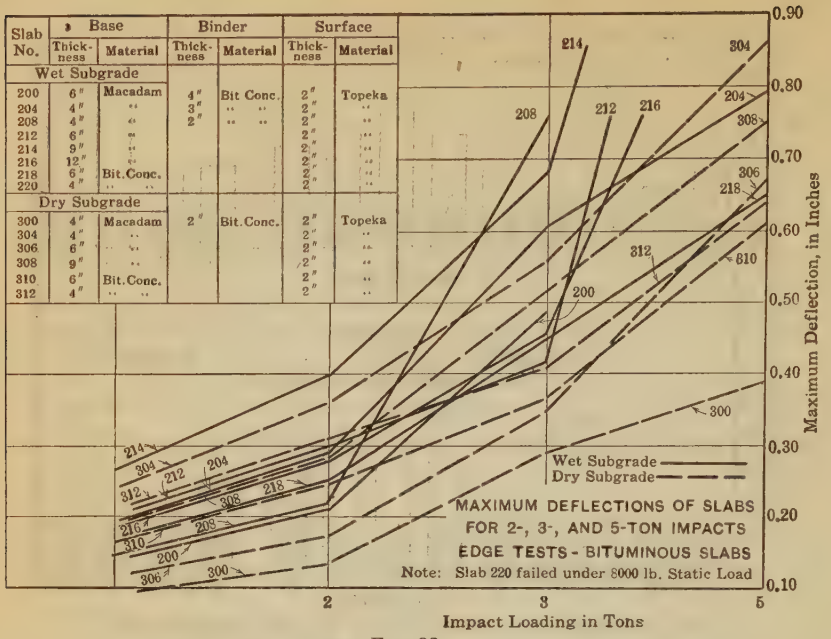
Slab No.
Base
Thick-ness
Material
Binder
Thick-ness
Material
Surface
Thick-ness
Material
Wet Subgrade
200 6″ Macadam 4″ Bit Conc. 2″ Topeka
204 4″ " 3″ " " 2″ " "
208 4″ " 2″ " " 2″ " "
212 6″ " 2″ " "
214 9″ " 2″ " "
216 12″ " 2″ " "
218 6″ Bit.Conc. 2″ " "
220 4″ " " 2″ " "
Dry Subgrade
300 4″ Macadam 2″ Bit.Conc. 2″ Topeka
304 4″ " 2″ " "
306 6″ " 2″ " "
308 9″ " 2″ " "
310 6″ Bit.Conc. 2″ " "
312 4″ " " 2″ " "
Maximum Deflection, in Inches
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
Wet Subgrade
Dry Subgrade
MAXIMUM DEFLECTIONS OF SLABS
FOR 2-, 3-, AND 5-TON IMPACTS
EDGE TESTS - BITUMINOUS SLABS
Note: Slab 220 failed under 8000 lb. Static Load
2
3
5
Impact Loading in Tons

FIG. 22.

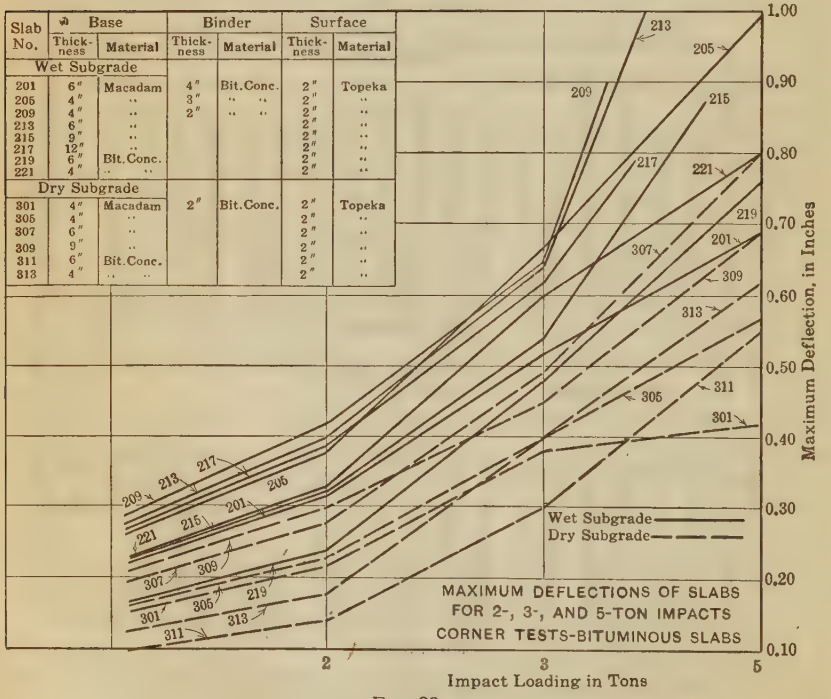
Slab No.
Base
Thick-ness
Material
Binder
Thick-ness
Material
Surface
Thick-ness
Material
Wet Subgrade
201 6″ Macadam 4″ Bit.Conc. 2″ Topeka
205 4″ " 3″ " " 2″ " "
209 4″ " 2″ " " 2″ " "
213 6″ " 2″ " "
215 9″ " 2″ " "
217 12″ " 2″ " "
219 6″ Bit.Conc. 2″ " "
221 4″ " " 2″ " "
Dry Subgrade
301 4″ Macadam 2″ Bit.Conc. 2″ Topeka
305 4″ " 2″ " "
307 6″ " 2″ " "
309 9″ " 2″ " "
311 6″ Bit.Conc. 2″ " "
313 4″ " " 2″ " "
Maximum Deflection, in Inches
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
Wet Subgrade
Dry Subgrade
MAXIMUM DEFLECTIONS OF SLABS
FOR 2-, 3-, AND 5-TON IMPACTS
CORNER TESTS-BITUMINOUS SLABS
2
3
5
Impact Loading in Tons

FIG. 23.

8.—There is very little evidence of "cushioning" due to bituminous tops at temperatures of 32° cent. (90° Fahr.), or less.

9.—Bituminous tops do not seem to add to the slab strength of a concrete base with the possible exception of the 4 and 6-in. bases on a dry sub-grade.

Pittsburg, Calif., Test Road

Brief mention should be made of the results of the service tests made on the specially constructed road at Pittsburg, Calif. This report has already been published in detail.* The test was initiated by the Columbia Steel Company of Pittsburg, and, later, became a co-operative investigation by the California Department of Public Works and the U. S. Bureau of Public Roads.

The types of sections used are shown in Fig. 24. The road was constructed entirely on an adobe sub-grade, which material in the past has always given considerable trouble. It should be noted that, in the construction of this road, the sub-grade was carefully prepared to a depth of 3 ft. The material, having been removed below grade to that depth, was carefully pulverized by harrowing and scarifying, sprayed lightly with water and compacted with a 12-ton three-wheeled roller. Each succeeding layer of 6 in. was treated in this way, resulting in a well-compacted homogeneous sub-grade.

Traffic was started November 9, 1921, in two lines, moving in opposite directions until December 24, 1921, when the broken condition of some of the slabs necessitated the use of a single line, which was continued until January 29, 1922. The gross loads on the trucks were increased from 7¼ tons to 13½ tons; the speed was about 12 miles per hour. Careful records were kept of the formation of cracks and broken areas, and auxiliary measurements included deflections of certain sections of pavements made by means of tunnels built under the pavement, temperature records, sub-grade tests, and tests on the concrete. The details of these investigations are too extensive to treat in this paper, but the results are given in Table 3, which shows a comparative rating of the different sections after 6 914 850 tons of traffic had gone over the pavement. In arriving at the unit comparison of total computed cost per mile, it was assumed that the cost of replacing broken areas of concrete would be three times that of laying the original concrete and that in all cases the expense of maintaining cracks would be 1 cent per lin. ft.

In general, it will be noted that the sections having a low percentage of failure are those which have been either stiffened or reinforced longitudinally. No doubt, if more of the transverse reinforcement in the various sections had been placed in a longitudinal direction, there would have been greater resistance to the traffic loads. The effectiveness of longitudinal stiffening ribs is to be noted particularly. Attention should be called also to the failure of Section I which had longitudinal stiffening ribs at the edge, but a thin center thickness with an unsupported center joint. The behavior of this

* Report of Highway Research at Pittsburg, Calif., 1921–22, Dept. of Public Works, California.

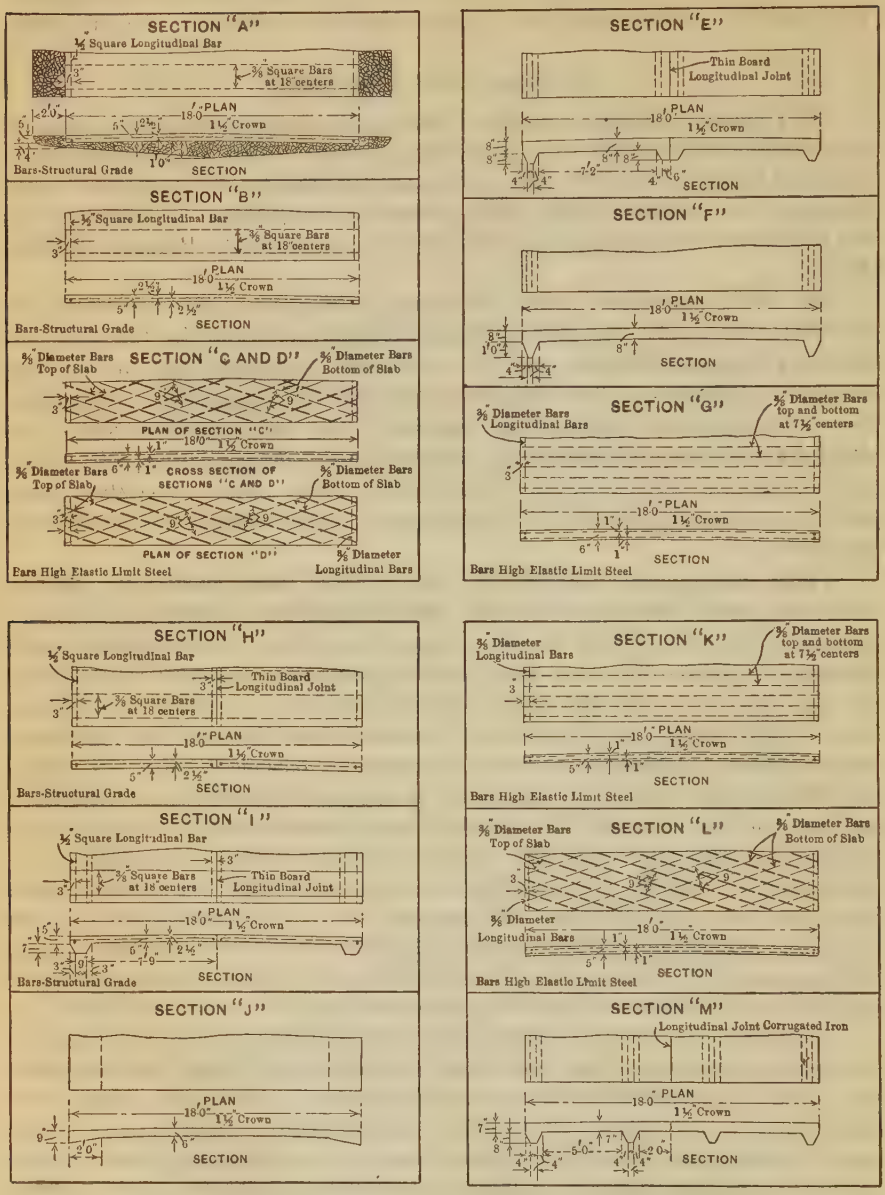

FIG. 24.—DETAILS OF VARIOUS SECTIONS OF PAVEMENTS FOR WOOD TESTS AT PITTSBURG, CALIF.

TABLE 3.—Comparative Pavement Ratings, Pittsburg, Calif., Test Road, June 30, 1922.
Traffic to June 30, 1922, 6 914 850 Tons.

Section.	Concrete per mile as actually laid, in cubic yards.	Steel per mile, in tons.	Total broken areas, in square feet per square yard of pavement.	Total linear feet of cracks per square yard of pavement.	Computed original cost per mile of pavement only.	Theoretical maintenance cost per mile.	Percentage of failure.	Total computed cost, maintenance plus original cost per mile.	Unit comparison of total computed cost per mile.
J	2 110	..	0.07	1.44	$31 439	$ 885	2.8	$32 324	1.000
D	1 845	55	0.36	2.01	33 271	4 205	12.6	37 476	1.159
C	1 798	55	0.45	2.56	32 549	5 152	15.8	37 701	1.166
K	1 722	69	0.53	2.66	32 643	6 049	18.5	38 692	1.197
F	2 719	..	0.00	1.11	39 426	117	0.3	39 543	1.223
M	2 574	..	0.22	0.70	37 323	2 810	7.5	40 133	1.242
E	2 806	..	0.00	0.60	40 687	63	0.2	40 750	1.261
G	2 127	69	0.56	2.04	37 902	7 291	19.2	45 193	1.398
L	1 740	55	1.56	2.29	31 659	16 705	52.8	48 364	1.496
B	1 590	20	2.99	3.91	26 207	26 534	100.0	52 741	1.632
H	1 760	24	3.09	3.91	29 176	30 464	100.0	59 640	1.845
I	1 953	24	3.12	3.01	32 139	33 743	100.0	65 882	2.038
A	1 834	20	4.08	3.74	35 427	48 576	100.0	84 003	2.599

section bears out the fact that the strength along the center joint must be equal to that along the edge of the pavement, unless provision is made for adequately carrying the load in part across to the adjacent slab. The effectiveness of stiffened edges is clearly shown, thus bearing out the results of the Bates Test Road, in Illinois* and corroborating the stress measurements already cited. The behavior of Section A is worthy of note. This section failed under comparatively little traffic after water had been turned into the side ditches. The water softened the sub-grade and the broken stone base then settled considerably as the sub-grade worked its way up through the voids in the stone. It is believed that a blanket layer of sand under the stone would have prevented this action.

Studies in Materials

Service tests have been conducted at Arlington on bituminous concrete and on Portland cement concrete, in order to make special studies of these materials.

Bituminous Concrete—Circular Track.—The increasing weight of traffic has resulted in the shoving and waving of bituminous pavements under certain conditions. Studies have been under way primarily to develop suitable tests which will be indicative of the degree of stability of bituminous mixtures under the action of traffic. A bituminous mixture at summer temperatures is apt to be quite plastic and may be permanently deformed under heavy loads. It is desirable, therefore, to make it sufficiently resistant at the highest pavement temperatures so that it will not deform under the heaviest intensity of wheel load.

Attempts are being made in the laboratory to develop a suitable test for determining the relative resistance to loads of bituminous mixtures, particularly at the highest temperatures, and these tests are being correlated with the service behavior of mixtures laid in a circular track at Arlington and subjected to motor-truck traffic. This track is approximately 185 ft. in diameter, built with a 6-in. gravel sub-base and a 6-in. concrete base. The finish of the base was made as uniform as possible, its smoothness approaching that of the ordinary concrete road. On this, twenty-seven sections of bituminous concrete mixtures were laid, in all of which trap-rock, Potomac river sand, limestone dust, and Mexican asphalt were used. While the pavement was being laid, a temporary wooden header was provided, but this was removed later for the construction of the test sections for concrete wear that now act as the outer curve. The entire circular roadway was given a seal coat of Mexican asphalt and trap-rock chips.

Traffic was started on the road in October, 1922, with a 3-ton truck loaded to capacity. Until June, 1923, when the truck had made 30 000 trips, none of the sections had moved, apparently. However, in June, the weather became suddenly much warmer and immediately the movement of certain sections under the traffic was noticed. During the warm weather, this movement became quite pronounced, and in addition to the 3-ton truck, a 5-ton truck loaded to capacity was also placed in operation until August 20, 1923,

* "Highway Research in Illinois", by Clifford Older, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1180.

when more than 49 000 trips had been made around the road, and a number of these sections had become so badly rutted and disintegrated that it was thought best to discontinue the test.

During the last three stages, the average air temperature at 2 P. M., was, as follows:

May 11 to June 25, 1923.....	81° Fahr.
June 25 to July 20, 1923.....	85.7° Fahr.
July 20 to August 17, 1923.....	83.5° Fahr.

From July 6 to August 16, temperatures were taken in one section at depths increasing by ½ in. Highest temperatures, about 2 P. M., were observed ½ in. below the surface, ranging from 95 to 132° Fahr., with a mean of 115° Fahr.

White lines were painted transversely across the pavement at the third-points of each section (Fig. 17). The diagrams, Fig. 25, give the maximum movement of these lines in the various sections. The greatest movement always occurred under the outer wheel, no doubt because of the increased wheel pressure produced by centrifugal force. It is hoped by the proper correlation of such service tests with laboratory studies that finally a suitable test for mixtures will be evolved, which will indicate their degree of stability under various conditions of temperature and loading.

Concrete Wear—Circular Track.—In order to study the effect of a number of different factors on the wear of concrete pavements, a great many sections were built in a circular track, 200 ft. in diameter, immediately adjoining the bituminous track, previously described. It was subjected to the wear of a special machine (Fig. 17), equipped with solid rubber tires loaded to 600 lb. per in. of width. The driving was done through two of the rubber-tired wheels at a speed of approximately 20 miles per hour. The auxiliary steel wheels merely served to guide the apparatus. The first stage of the test consisted of 60 000 runs of this machine and the second stage of about 10 000 runs over a new path, but with two of the wheels equipped with skid chains. The results after 10 000 runs of the wearing device equipped with skid chains are shown in Fig. 26.

Data on the various sections used are given as follows, Groups I to VII, inclusive:

Group I.—Quality of Crushed Stone as Coarse Aggregate.—

Mix.....	1:1½:3
Fine aggregate.....	Potomac River sand
Slump.....	1 in.
Time of mix.....	1 min.

Section 1.—Martinsburg limestone, laid August 14, 1922
" 2.—Bound Brook, N. J., trap, laid August 14, 1922
" 3.—Sioux Falls, S. Dak., quartzite, laid August 15, 1922
" 4.—Llano, Tex., granite, laid August 15, 1922
" 5.—Burnet County, Tex., limestone, laid August 15, 1922
" 6.—Sandstone County, Minn., sandstone, laid August 15, 1922
" 7.—Esserville, Va., Sandstone "B", laid August 15, 1922
" 8.—Bellevue, Ohio, limestone, laid August 16, 1922
" 9.—Rogers, Mich., limestone, laid August 16, 1922
" 10.—Stone City, Iowa, limestone, laid August 16, 1922

MAXIMUM MOVEMENT 80″ 60″ 40″ 20″	ASPHALT		GRADING OF TOTAL AGGREGATE								SEC. No.	MIX. No.
	Pen	%	R 1¼	R ⅜	⅜-¼	¼-10	10-40	40-80	80-200	P-200		
	86	4.5	1.2	20.5	45.4	9.5	8.6	6.6	5.3	4.1	17	1
	76										18	2
	65										19	3
	55.5										16	4
	75	4.9	1.6	23.6	34.0	9.9	13.0	9.8	6.5	3.2	13	5
	64.5										14	6
	55.5										12	7
	46										15	8
	75.5	6.0	1.5	18.7	31.6	7.0	16.5	13.8	8.7	3.7	3	9
	65										2	10
	55										1	11
	45.5										4	12
	76	6.3		9.6	23.8	9.4	23.5	19.0	10.6	4.1	23	13
	65										21	14
	55										20	15
	45										22	16

MAXIMUM MOVEMENT 8″ 6″ 4″ 2″	ASPHALT		GRADING OF TOTAL AGGREGATE								SEC. No.	MIX. No.
	Pen	%	R 1¼	R ⅜	⅜-¼	¼-10	10-40	40-80	80-200	P-200		
	75.5	6.0	1.5	18.7	31.6	7.0	16.5	13.8	8.7	3.7	3	9
	65										2	10
	55										1	11
	45.5										4	12
	75.5	5.2	6.9	26.0	25.7	11.7	15.5	10.8	7.0	3.3	6	17
	55										7	18
	45.5										5	19
	55.5	6.3		16.4	30.4	12.3	19.0	12.5	6.7	2.7	11	20
	75	6.0		18.2	27.5	7.7	18.0	15.2	9.6	3.8	9	21
	55.5										8	22
	46										10	23

MAXIMUM MOVEMENT 8″ 6″ 4″ 2″	ASPHALT		GRADING OF TOTAL AGGREGATE								SEC. NO.	MIX. NO.
	Pen	%	R1¼	R¾	¾-¼	¼-10	10-40	40-80	80-200	P200		
	55	6.0	1.5	18.7	31.6	7.0	16.5	13.8	8.7	3.7	1	11
	55	5.2	6.9	26.0	25.7	11.7	15.5	10.8	7.0	3.3	7	18
	55	6.3		16.4	30.4	12.3	19.0	12.5	6.7	2.7	11	20
	55.5	6.0		18.2	27.5	7.7	18.0	15.2	9.6	3.8	8	22
	55	4.9		20.5	24.8	9.9	21.8	16.1	4.7	2.2	25	24
	55	5.0	4.7	24.6	23.5	12.4	19.1	13.6	4.3	2.5	24	25
	55	7.5	4.3	11.9	28.7	9.5	11.9	15.1	13.6	9.3	26	26
	55	7.3		9.5	31.6	5.4	7.6	18.8	18.6	8.5	27	27

FIG. 25.—SHOVING OF ASPHALT PAVEMENTS.

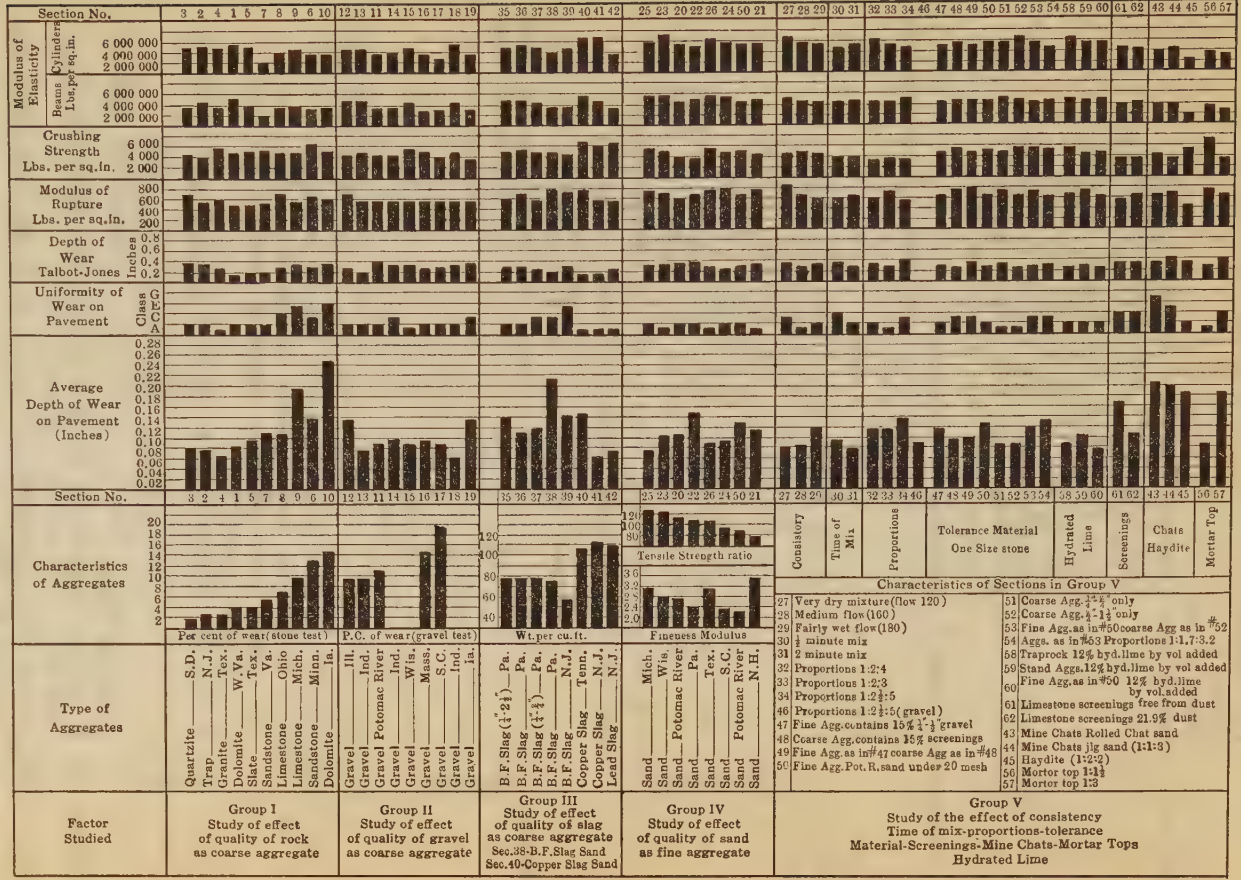

FIG. 26.—CHART OF RESULTS OF WEAR TESTS, CONCRETE PAVEMENTS, AFTER 10 000 RUNS, WHEELS EQUIPPED WITH SKID CHAINS.

Group II.—Quality of Gravel as Coarse Aggregate.—

Mix.....1:1½:3
Fine aggregate.....Potomac River sand
Slump.....1 in.
Time of mix.....1 min.

Section 11.—Gravel from Potomac River, laid August 16, 1922
" 12.— " " Plainfield, N. J., laid August 16, 1922
" 13.— " " Williamsport, Ind., laid August 17, 1922
" 14.— " " South Bend, Ind., laid August 17, 1922
" 15.— " " Wausau, Wis., laid August 17, 1922
" 16.— " " Ludlow, Mass., laid August 17, 1922
" 17.— " " Wateree, S. C., laid August 17, 1922
" 18.— " " Evansville, Ind., laid August 18, 1922
" 19.— " " Hancock, Iowa, laid August 18, 1922

Group III.—Quality of Sand as Fine Aggregate.—

Mix.....1 : 1½ : 3, using 1½-in.-¼-in. Martinsburg limestone
Slump.....1 in.
Time of mix.....1 min.

Section 20.—Sand from Potomac River, laid August 18, 1922
" 21.— " " Concord, N. H., laid August 18, 1922
" 22.— " " Point Marion, Pa., laid August 18, 1922
" 23.— " " Wausau, Wis., laid August 21, 1922
" 24.— " " Pon Pon, S. C., laid August 21, 1922
" 25.— " " St. Clair River, Mich., laid August 21, 1922
" 26.— " " Colorado River, Texas, laid August 21, 1922

Group IV.—Effect of Consistency, Time of Mix, and Proportioning.—

Proportions: As indicated, using Martinsburg limestone, 1½ in.-¼ in., as coarse aggregate and Potomac River sand as fine aggregate.

Section 27.—1 : 1½ : 3 mix...½-in. slump...1-min. mix, laid August 21, 1922
" 28.— " " ...1- " " " laid August 23, 1922
" 29.— " " ...2- " " ...1- " " "
" 30.— " " ...1- " " ...½- " " "
" 31.— " " ...1- " " ...2- " " "
" 32.—1 : 2 : 4 " ...1- " " ...1- " " "
" 33.—1 : 2 : 3 " ...1- " " ...1- " " "
" 34.—1 : 2½ : 5 " ...1- " " ...1- " " laid August 25, 1922

Group V.—Quality of Slag as a Coarse and Fine Aggregate.—

Proportions: 1 : 1½ : 3 by volume, using Potomac River aggregate, except where otherwise indicated; slump, 1 in.; time of mix, 1 min.

Section 35.—B. F. slag, Pottstown, Pa., 2½ in.-¼ in., laid August 25, 1922
" 36.— " " " 1½ " -¼ " "
" 37.— " " " ¾ " -¼ " "
" 38.— " " " 1½ " -¼ " slag sand, laid August 25, 1922
Section 39.—B. F. slag, Wharton, N. J., 1½ in.-¼ in., hand-picked, laid August 25, 1922
Section 40.—Copper slag, Copperhill, Tenn., 1½ in.-¼ in., slag sand, laid August 28, 1922
Section 41.—Copper slag, Perth Amboy, N. J., 1½ in.-¼ in., laid August 28, 1922
Section 42.—Lead slag, Perth Amboy, N. J., 1½ in.-¼ in., laid August 28, 1922

Group VI.—Quality of Miscellaneous Aggregates.—

Proportions .....As indicated.
Time of mix.....1 min.
Slump .....1 in.

Section 43.—Chert chips (½ in.-¼ in.) rolled chat sand, 1 : 1½ : 3 mix, laid August 28, 1922

Section 44.—Chert chips (mine-run) jig sand, 1:1 : 3 mix, laid August 28, 1922

Section 45.—Haydite (burnt clay) coarse and fine, mix changed to 1 : 2 : 2 after trial batch, laid August 28, 1922

Section 46.—Gravel (Potomac) and Potomac sand, 1 : 2½ : 5 mix, only specimens made were three compression cylinders, laid August 28, 1922

Group VII.—Effect of Tolerance Material, One-Size Stone, One-Size Sand, Screenings in Place of Sand, Mortar Tops, and Hydrated Lime.—

Proportions .....1 : 1½ : 3, except as noted.
Time of mix.....1 min.
Slump .....1 in.

Section 47.—Martinsburg limestone, Potomac River sand, with 15% gravel, ½ in.-¼ in., laid August 29, 1922.

" 48.—Martinsburg limestone, with 15% screenings free from dust, Potomac River sand, laid August 29, 1922

" 49.—Martinsburg limestone, with 15% screenings free from dust, Potomac River sand, laid August 29, 1922

" 50.—Martinsburg limestone, Potomac River building sand, laid August 30, 1922

" 51.—Martinsburg limestone, ¼ in.-¾ in. material only, Potomac River sand, laid August 30, 1922

" 52.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River sand, laid August 30, 1922

" 53.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River building sand, laid August 30, 1922

" 54.—Martinsburg limestone, ¾ in.-1½ in. material only, Potomac River building sand, 1 : 1.7 : 3.2, laid August 30, 1922.

" 55.—Martinsburg limestone, Martinsburg limestone screenings, $\frac{1}{16}$ in.-¼ in. material only, laid August 30, 1922

" 56.—Mortar top, Potomac River sand, 1 : 1½, laid August 31, 1922

" 57.—Mortar top, Potomac River sand, 1 : 3, laid August 31, 1922

" 58.—Trap-rock dresser, Junction, Wis., Potomac River sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 59.—Martinsburg limestone, Potomac River sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 60.—Martinsburg limestone, Potomac River building sand, with 12% (by volume) hydrated lime, laid August 31, 1922

" 61.—Martinsburg limestone, Martinsburg limestone screenings, free from dust (4%), laid September 1, 1922

" 62.—Martinsburg limestone, Martinsburg limestone screenings, 21.9% dust, laid September 1, 1922

Miscellaneous Investigations

In addition to the investigations previously described, a number of other studies are under way in the U. S. Bureau of Public Roads, or in various other institutions in co-operation with the Bureau, covering different phases of the design of highways. For example, the University of Maryland is deter-

mining the elastic properties of concrete under repeated stress and is likewise making a study of Poisson's ratio for concrete. The first study will furnish results of value in connection with the selection of a proper unit stress and the second, results which may be used after the development of the proper design theory.

Similar studies are being made at Purdue University relating to fatigue tests of concrete and also the stress produced in concrete by moisture changes.

The University of Georgia is making some special studies of sand-clay and top-soil roads.

The Iowa State College is working on problems of highway bridge impact and loads on highway culverts.

Studies are also being made by other organizations, in co-operation with the U. S. Bureau of Public Roads, dealing with other phases of road design. It is believed that the data now being obtained will finally result in the use of more scientific and more rational methods of road design than have been found possible in the past.

DISCUSSION

PRÉVOST HUBBARD,* AFFILIATE, AM. SOC. C. E.—The question has been raised as to the relation of smoothness of the concrete base to displacement of an asphalt top under traffic. The speaker has inspected the Arlington experiments (but not since any of the sections were taken up), and is interested in the progression of movement in the sections which shoved; whether the shoving apparently progressed from the top down or whether it was most pronounced at the point of contact with the base. He understands that metal plugs were buried at different points in the mixture for the purpose of determining the relative movements at different depths. It is thought this will be an important matter to determine, and may indicate whether or not the smoothness of the base has any effect upon shoving.

Another question is in connection with the impact test results. The basis of comparison of resistance of the different types of slabs appears to be the number of pounds of impact required to cause failure. The question arises as to whether this is the proper gauge to use for different classes of materials. The speaker believes it has been ascertained that a 5-ton truck running at a speed of 12 miles per hour, and falling from a height of 2 in., delivers an impact of about 28 700 lb. on a gravel road and about 36 800 lb. on a concrete road. This means that, the conditions of test being equal, the number of pounds of impact varied. Now, in comparing different types and designs of road slabs or road structures, on a basis of impact pounds delivered, will not this mean that the conditions causing the same number of impact pounds on the different types may be different—in other words, more severe in one case than another—and, if that is so, is it proper to make the basis of comparison entirely on impact pounds? Comparison should be made as regards the behavior of the different types subjected to the same conditions of test rather than as regards the intensity of blow causing failure.

Has any attempt been made to determine the effect of fatigue on the rigid types of construction under impact as has been done under static or suddenly applied loads? Is it not possible that fatigue may play a more important part in the resistance of rigid structures to impact than in their resistance to repeated static loading?

JACOB FELD,† ASSOC. M. AM. SOC. C. E. (by letter).—The design of highways, like that of many other structures, has been brought to a more scientific basis by logical deductions from numerous large sized experiments, of which those reported by the author are probably the most far-reaching and the best planned. As in many other engineering designs, trouble in highways arises not in the structure itself, but in the design of the foundation. The infinity of possibilities as to the nature of soils and the large number of seemingly small, but really important, factors which enter into any investigation of materials not readily classed as solids or gases, makes any design in which forces or reactions from such materials have to be considered, indeterminate

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† Structural Engr., New York, N. Y.

in many ways. According to the best present practice, the method of design, as for all truly indeterminate structures, is to find one possible solution of equilibrium compatible with the conditions of the problem and to trust to the Moseley Principle of Least Resistance that the true solution of equilibrium will result in stresses no larger than those found. To determine the worst (or end) conditions which may govern the design, full sized experiments are required in which the factors are varied from one extreme to the other.

No logical design can be expected until the materials are scientifically classed into groups containing those which act similarly under similar conditions. Great progress has been made in the classification and standardization of road materials in the past few years; the sub-grade has not received equal attention and is now the weak link in the chain of highway design.

The investigations of the Special Committee of the Society on the Bearing Value of Soils, etc., have proven the importance of the colloid content in the action of soils. This fact may be expressed in another way—that the absolute size of the soil particles is an important factor. (A material is called colloidal when its individual particles are from 0.08 mm. to molecular dimensions in size; the hydrogen molecule is approximately 0.00000016 mm.). Until quite recently, the relative grain size or gradation received major attention.

The importance of the absolute size of particles is evident from the list of tests of the U. S. Bureau of Public Roads to distinguish and define the various properties of sub-grade materials. The first test, mechanical analysis, determines the percentage of materials of each size. The remainder of the list reads like a description of the standard tests in physical chemistry to determine the properties of a colloidal substance. There is not much hope, however, in this type of analysis, for even a cursory study of colloid chemistry discloses the fact that, although there is a large mass of empirical data, there has been little scientific progress. In addition, chemists have naturally devoted their attention chiefly to the action of chemicals on colloids. A little has been done in the matter of temperature changes; but very few experimenters have investigated the effect of loads, impact, vibration, and similar factors. Further, practically all research in this field has dealt with colloids, *per se*, and not with materials containing a colloidal constituent. Throughout this work discrepancies and unexplained contradictions in results exist.

The recent work in soils as an engineering material marks a departure from the customary research in materials of construction. All design methods at present are purely empirical or are based on data checking a definite theory. The first work in any field was an attempt at a complete solution of the problem—the effect of definite forces on the material under consideration. Later, the investigation was broadened into a study of the materials themselves. For example, the methods of designing metallic structures were known long before any investigation was begun into the nature of iron or non-ferrous metals and into the methods by which these metals could be controlled to give certain desirable characteristics. After a few experiments concerning the proper design of foundations, the distribution of loads through soils, etc., the research work is now practically entirely devoted to a study of the nature of soils. This

trend will probably result in a more correct first solution, but meanwhile there is nothing tangible to guide the designer. For example, it is assumed that the soil reaction beneath a column footing is uniform. It is known that this is far from exact; yet, what is a more correct assumption?

Several points in the author's discussion of the sub-grade are of especial interest to the student of colloids. The importance of soil shrinkage is stressed. Can the abnormal shrinkage of the sub-grade of concrete roads be caused by the action on the colloids in the soil of the chemicals leached from the concrete by the excess water? Clay, for instance, is easily coagulated by lime. A control of the colloids should eliminate part, at least, of the soil shrinkage below pavements, just as the shrinkage of farm land is controlled by proper chemicals.

That part of the paper dealing with the influence of the bearing area on the bearing value of soils is the first available experimental data on this phase of the soil problem. In 1923, the writer investigated the literature regarding the proportionality of bearing area and supported load, and found practically nothing. To be exact, the only experiments on record were a few large tests on some building foundations, the New York Municipal Building especially, some small tests by Yankovsky, and a few incidental tests by Crosthwaite and others in connection with similar work. These references, with a theoretical paper by Keck, in which the proportionality of bearing area and pressure is experimentally proved for several solid materials and also claimed for sands and soils (although no experimental data are given) constituted the only literature that a rather careful search revealed.

The author mentioned the fact that the bearing value of soils, especially sands, is highest for a definite moisture content, usually a moist condition. This is associated with the fact that the density of soils is a maximum at some low moisture content, that is, a definite quantity of soil will occupy the least volume when it contains a definite small percentage of moisture. An increase or a decrease in moisture causes an increase in volume. In explanation, this is usually assumed to be the effect of the surface tension of the water films covering all soil particles, drawing these particles together. The proper quantity of water covers each particle with no excess. Additional water fills the pores and, as water resists compression, tends to resist the decrease of volume to a minimum. Too little water permits grains to touch each other, corner to corner, and not arranged in closest possible relation to each other.

The author is to be congratulated on his clear presentation of a wealth of empirical data covering so many features of highway design. The writer has discussed only that section of the paper dealing with the foundation in which he is especially interested.

A. S. Woodle, Jr.,* M. Am. Soc. C. E. (by letter).—No method has yet been formulated for the rational design of a concrete slab supported on a sub-grade of a heterogeneous, non-elastic material. All such structures are designed empirically, by the rules of experience, and researches like the present one are valuable as tending to place their design on a more rational basis.

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The fundamentals involved in the design of any structure are well stated by the author as follows: (1) Correct evaluation of the forces which are to be resisted; and (2) correct proportioning of the structure to resist those forces.

The whole difficulty in the design of a concrete road or pavement lies in the first of these two problems. As this difficulty is caused entirely by the bewildering complexity of the many variable properties of the average sub-grade, the author very naturally has attacked the problem by endeavoring to learn experimentally something of these little known properties of the various materials which may make up the sub-grade.

A logical method of approach to this problem would be first to simplify it by eliminating as many of the unknown variables as possible; the evaluation of the forces at work in this simple case would increase the facilities for attacking the more complicated general case.

Consider, for example, a concrete floor for a warehouse in the tropics, on filled ground, to be held in place by retaining walls. The effect of temperature may be neglected and also that of moisture, thereby simplifying the problem considerably.

Such a floor will be stressed most severely if an unloaded part is immediately adjacent on each side to one that is fully loaded. Suppose, for example, that the warehouse floor is entirely covered with a load of w lb. per sq. ft., except for an aisleway through the middle, L ft. wide. Then, the concrete slab at certain points of this unloaded part will receive the greatest possible stress.

Consider a strip 1 ft. wide across the unloaded part. This strip (Fig. 27) may be pictured as an inverted beam restrained at the ends, carrying a load, varying from a maximum of w lb. per sq. ft. at each end to a minimum at the center, designated by cw lb. per sq. ft., c being a factor between 0 and 1. It is evident that c will vary from 0 in the case of an absolutely rigid sub-grade (like bed-rock), to almost 1, in the case of a soil permitting considerable settlement. It should certainly be possible by proper tests and experiments to determine the value of c for any given set of conditions; and granting that this is so, the following line of reasoning may be suggested as an approximate solution of the problem.

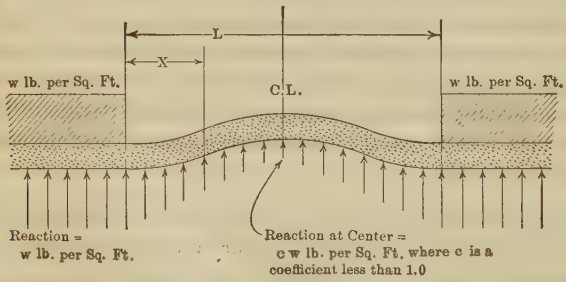

FIG. 27.

If the rate of loading of this beam be considered as a straight line, change from w lb. per sq. ft. at each end to cw lb. per sq. ft. at the center; the assump-

tion should be close to the truth. This would give the rate of loading at any point, X, in the span, L, as:

$$w\, c_x = w - \frac{2\, x\, w}{L}\,(1 - c) = \frac{d^4\, y}{d\, x^4} \times E\, I$$

Integrating this expression twice will give an expression for the bending moment and substituting in this for x, the value, 0, as the point of greatest bending moment, will give:

$$\text{Maximum bending moment} = \frac{w\, L^2}{192}\,(1 - c) - \frac{w\, L^2}{32}$$

This expression is not exact, as to obtain the constant for the second integration, the point where the bending moment is 0 was assumed to be at $\frac{L}{4}$, which is only an approximation. From this expression the floor-slab may be designed, provided the value of c can be deduced.

The value of c, it is evident, can be determined only by experiment. It will range between 0 and 1, depending on: (1) The compressibility of the soil; and (2) the stiffness of the concrete slab.

The writer believes that it would be possible by a systematic series of tests to deduce the laws by which the compressibility of the soil varies with known properties (such, for example, as the density) and thus approach a rational method of determining the value of c for any given quality of sub-grade material.

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Paper No. 1558

THE COLORADO RIVER PROBLEM*

By William Kelly,† M. Am. Soc. C. E.

With Discussion by Messrs. J. C. Allison, Frederick H. Fowler, E. C. La Rue, Arthur P. Davis, C. E. Grunsky, F. E. Weymouth, G. E. P. Smith, Louis C. Hill, C. S. Jarvis, E. W. Lane, Raymond A. Hill, E. B. Debler, and William Kelly.

Synopsis

So much has been written and spoken on the subject of the Colorado River in the past few years that perhaps the greatest need at present is an impartial digest of the facts and an estimate of the probabilities. This paper attempts to give both; it also presents some as yet unpublished studies of flood protection, water supply, irrigable areas, and power demand which are important factors in any study of the Colorado. Finally, it sets forth the salient features of a comprehensive scheme of development and points out the lines on which activities of the Federal Government should be directed.

Introduction

The Colorado has its sources in the melting snows of the mountains of the Continental Divide. Its drainage basin has an area of nearly 250 000 sq. miles and covers parts of seven States. Its delta is in Mexico through which it flows to the Gulf of California. Development, mostly irrigation, has gone forward to the present with little or no thought of a comprehensive use of the entire river. Fortunately, what has been done will not interfere materially with the full use of this great water resource, but in view of the large projects under consideration it is important that a general scheme of development be

* Presented at the Annual Convention of the Society, Pasadena, Calif., June 19, 1924.
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adopted and that supervision be exercised to require future developments to conform thereto. Such supervision can be exercised under existing laws by co-operation between the Federal Government and the seven interested States. Although many additional data on stream flow, location, and character of irrigable lands, dam sites, etc., are needed before details of the ultimate development can be determined, sufficient information is available to determine the general scheme in so far as that is necessary for the consideration of projects now advocated.

Scheme of Development

The essential features of a comprehensive plan of development for the Colorado are determined by topographic conditions. The Upper Basin, which lies above the junction of the main river with the Green River in Utah, has possibilities of development for both irrigation and power; the total area that can be irrigated probably does not exceed 4 500 000 acres, of which about one-third is now irrigated. This area is from 4 000 to 8 000 ft. above sea level and use of water for irrigation will always be limited by climatic conditions. The depletion of water supply by irrigation in this area will probably never exceed 4 500 000 acre-ft. To this should be added diversions out of the Basin which may reach 500 000 acre-ft.

In the Upper Basin there are many possibilities for power development, the most promising of which are on the Green between Green River, Wyo., and the junction of the Green with the Colorado. Interference between these power developments and irrigation should be given careful consideration before the developments are undertaken. Such consideration is assured by the fact that practically all such projects involve the use of public lands and thereby come under the jurisdiction of the Federal Power Commission.

The Middle or Canyon Section extends from the junction of the Green to a short distance below Needles, Calif. Irrigation in this section will never be important, being limited to a few inter-canyon valleys of doubtful feasibility. The importance of this section is due to the 4 000 000 h.p. it can produce. If water is reserved for up-stream irrigation, and re-regulation is provided for irrigation below, there will be no interference with irrigation by power in this section.

Below Needles, including Mexico, there are more lands that can eventually be irrigated than there will be water to serve. Power possibilities in this section are small and will be incidental to irrigation development. The Gila River enters the Colorado just above Yuma, Ariz. The development of its water resources has no bearing on the plan for the Colorado, except in the matter of flood protection of the delta region.

Permanent settlement of the Colorado Basin will be largely dependent on irrigation. It is probable that in the immediate future power will have a greater value than irrigation, but as power can be obtained from other sources than the river, it should not be allowed to curtail the ultimate irrigation development.

Although it is impossible to predict the rate at which either irrigation or power will develop, it is certain that there are more than enough irrigable lands

to use all the water available and that all the power that can be developed in the main section of the river will be needed. The plan of development, therefore, should provide that:

1.—Losses from evaporation should be kept to a minimum.

2.—All available head should be used for power.

3.—Storage for regulation of flow should be located above the Canyon Section so that the equated flow can be used through the greatest practicable head.

4.—Storage in and below the Canyon Section should be limited to that necessary for re-regulation of flow for irrigation in the Lower Basin plus such quantity as is essential for immediate flood relief of existing developments in the Lower Basin. Failure to conform to this provision will mean ultimate duplication of storage capacity and consequent curtailment of irrigation due to unnecessary evaporation losses.

There are reservoir sites in the Upper Basin of sufficient capacity to provide regulation of flow. The first cost of such reservoirs per acre-foot of storage will be more than that for sites in the lower Canyon Section, but they will be worth as much for irrigation, three times as much for power, and, on account of regulated flow, will make possible material savings in cost of power dams built below them. Moreover, they can be developed successively as needed so that if interest on investment is considered, their ultimate cost may not be much greater, even on an acre-foot basis.

Present developments in the Lower Basin are subject to damage by floods. The development of storage in interest of power will relieve the flood menace in part. Full protection, in so far as it can be obtained by storage, requires that the storage capacity be available at the beginning of each flood season. Storage for power on the other hand should be operated to keep reservoirs as full as practicable at all times. The United States Bureau of Reclamation estimates that ultimately 4 000 000 acre-ft. will suffice for flood protection. As will be shown hereafter, 4 000 000 acre-ft. is all that is justified at present. Flood storage to afford the greatest benefit should be as far down stream as practicable.

Flood Protection

The Lower Basin is menaced by floods from both the Gila and the Colorado Rivers. The Gila floods are produced by winter rains and generally occur between November 30 and March 1. They reach a maximum flow of about 200 000 sec-ft. (approximately the flow over Niagara Falls). They are extremely flashy and, therefore, produce higher flood stages and greater velocities than the more deliberate floods of the Colorado. As long as the Gila remains uncontrolled, it will determine the height to which levees must be maintained at and below Yuma.

There is general agreement that, although the Gila, until controlled, will require maintenance of levees at present heights, it will never seriously inundate the Imperial Valley, because it discharges no water for a large part of the year. There are several irrigation and power projects under consideration,

which, if completed, will materially reduce the magnitude of Gila floods. No proposal is being seriously advanced that the Federal Government should undertake control of the floods of the Gila River.

FIG. 1.

The main floods in the Colorado are due to melting snow in the upper regions of the river. About 86% of the total run-off at Yuma comes from north of Arizona. These floods begin as early as March and may last into July, and their maximum flow is about the same as that of the Gila, but they con-

tain vastly more water. The floods in the Lower Colorado affect three existing projects, namely, Palo Verde, Yuma, and Imperial Valley, both in the United States and in Mexico.

Palo Verde.—The Palo Verde Project (see Fig. 1) is on the west side of the river in Riverside and Imperial Counties, California, and is protected by a levee 28½ miles long. The engineer of the Project states that the lands would be flooded if no levees were provided when the flow exceeds 50 000 sec-ft. and that bank protection is required when the flow exceeds 35 000 sec-ft. The total area irrigable is 78 000 acres, of which 44 000 acres were irrigated in 1922.

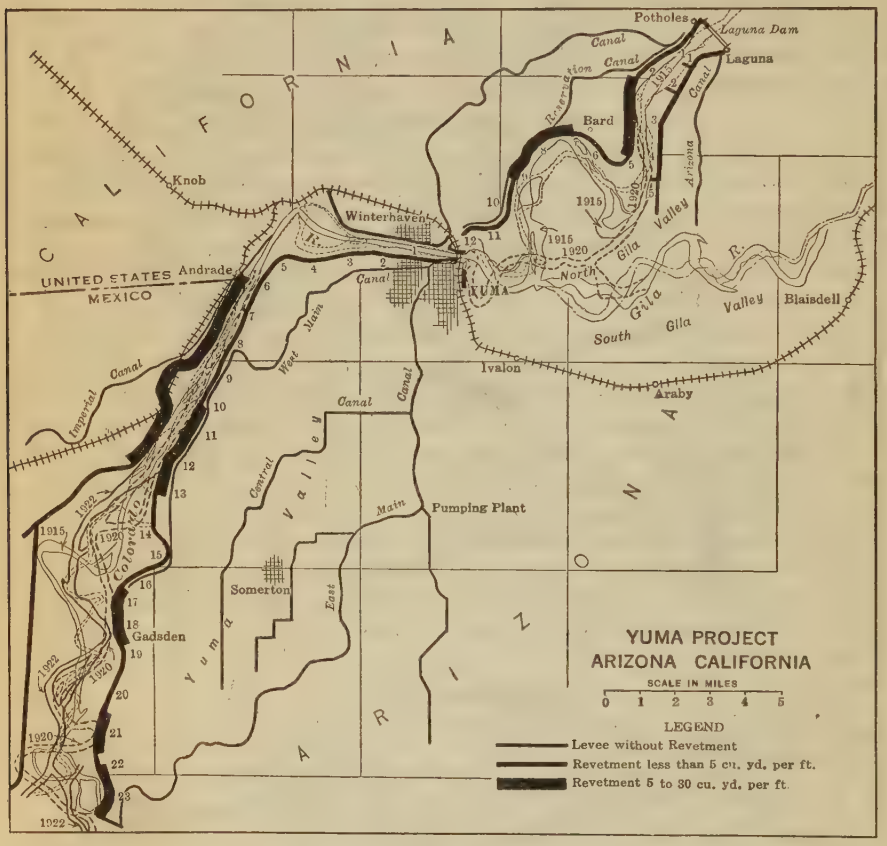


FIG. 2.

Yuma.—The Yuma Project (see Fig. 2) has a total irrigable area of 114 000 acres, 55 700 acres of which were irrigated in 1922. Of this land, 15 000 acres lie in the Yuma Indian Reservation on the California side between Laguna Dam and a point opposite Yuma. These lands were subject to overflow at extreme high water and have been protected by a levee about 12 miles long, extending from Laguna Heading to the Southern Pacific Railroad track near Yuma. The Potholes Branch of the Southern Pacific Railroad is located on

this levee. The levee crosses several old meanders of the river and, consequently, is subject to attack by it at nearly every high water. Bank protection has been necessary to maintain the levee, and the bank is now revetted practically throughout its entire length. The levee has been breached by the Colorado River floods once (in 1913) and was overtopped at its lower end by the Gila flood of January, 1916.

Another 54 000 acres lie in Yuma Valley on the Arizona side between the Town of Yuma and the Mexican Boundary. These lands were subject to overflow at high water and have been protected by a levee 25 miles long, extending from Yuma to the Mexican Boundary. A Government railroad has been built on this levee in order to facilitate its maintenance. For 12 miles below Yuma, the river flows in a narrow and relatively straight channel with fairly stable banks and has never given much flood trouble. Below that point, it shows a determination to repeat its meandering into the land now occupied by the Project. Since 1909, the river has made direct attacks on this levee at 12, 16, and 24 miles below Yuma, necessitating heavy expenditure for bank protection. The levee has never been overtopped, but has twice failed from undercutting, once from the Gila flood of January, 1916, and once from the Colorado River flood of 1920. The construction and maintenance of the levees of the Yuma Project cost $3 234 470. The annual maintenance for the past six years has averaged $86 420. The remainder of the lands in the Project, about 45 000 acres, are on the mesa east of Yuma Valley, well above high water.

The Yuma levees are nearer the river than has been found to be economically practicable elsewhere. They encroach on a part of the valley that the river has used in the past and will try to use in the future. They can be maintained in their present location only by heavy bank protection, and maintenance expense will be high until the banks are sufficiently protected whether or not flood storage is provided. (See Fig. 2.)

Imperial Valley.—The Imperial Valley (see Fig. 3) comprises about 515 000 acres of irrigable land in California and 255 000 acres in Mexico. During 1923, about 400 000 acres of the area in California and about 170 000 acres in Mexico were irrigated. All this highly productive area lies below the low-water level of the river and much of it below sea level. Water is diverted for irrigation at Rockwood Heading about a mile up stream from the International Boundary line. The main canal follows generally an old flood channel called Alamo River. The Valley was first protected from flood waters by the head-works of the canal and by an embankment on the river side of the canal. This embankment now forms a secondary line of defense. It was supplemented by a stronger line of defense consisting of the levees of the California Development Company, Sais Levee, extending from the canal intake south along the river for about 10½ miles, thence bearing west for about 16 miles, and the Volcano Lake Levee north of Volcano Lake from high ground west of the Valley to the Inter-California Railway Company embankment and the Imperial Canal near Bataques Station.

In 1905, the Imperial Canal had silted, for about 4 miles below the Heading, to such an extent as to threaten a water famine during the low-water period. In order to get water into the canal, a cut was dredged to turn the river into it

at a point about 7 miles below the Heading. An early flood season defeated attempts to close the dredged cut and the entire flood flow passed into the Valley. It was not until 1907 that efforts to turn the river back into its old channel were successful. The closing of this break cost about $2 000 000. In 1909, the river left its old channel at a point about 29 miles below Yuma and turned west through Bee River into Volcano Lake. This necessitated the extending of the Sais Levee to an intersection with the Volcano Lake Levee and of raising and reinforcing the Volcano Lake Levee. Almost the entire silt

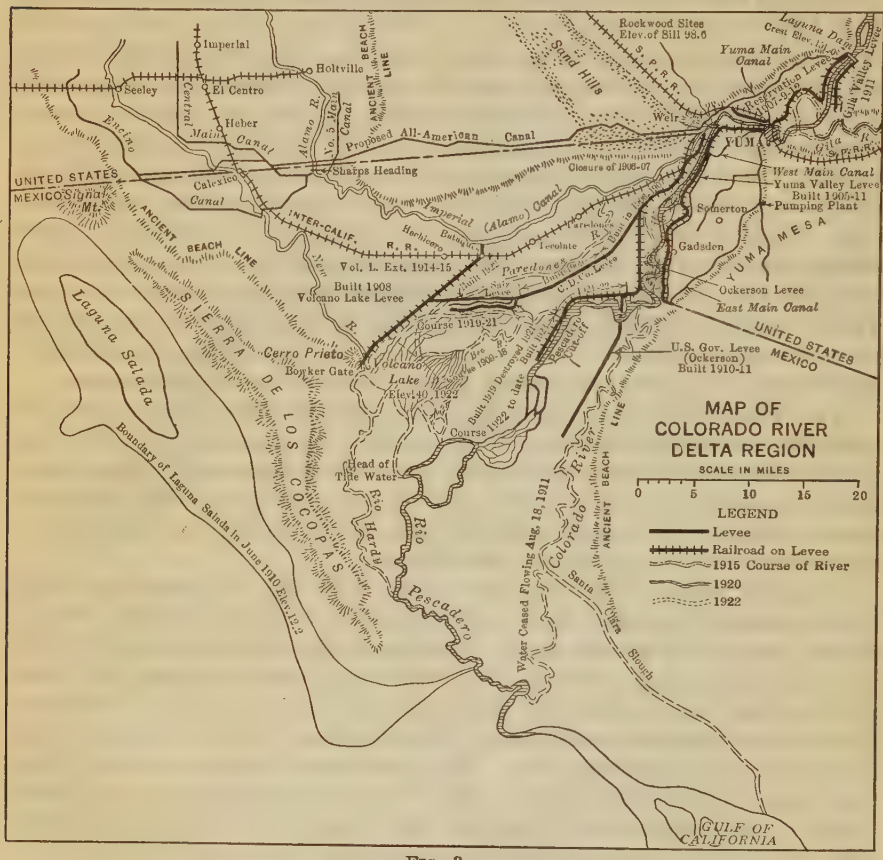


FIG. 3.

load of the river was deposited in Volcano Lake and its bed was raised at a rate that made it difficult to keep the Volcano Lake Levee above it. In 1911, the Federal Government provided $1 000 000 to relieve the situation. The funds were used to build the Ockerson Levee in an attempt to put the river back into its old channel. (See Fig. 3.) The attempt was unsuccessful. In 1922, the Pescadero Cut-off was built and the river successfully turned from Bee River and Volcano Lake into Pescadero River. As a part of the Cut-off project, the Ockerson Levee was repaired and extended about 8 miles along the north bank of Bee River, thence south across the river and down the west bank of the

cut-off channel. This new levee now constitutes the main line of defense. No alarm is now felt for the levee of the California Development Company from the intake down to the Ockerson Levee, because, in its present condition, except during Gila floods, the river never gets over its banks along this stretch. Even if the levee were breached, as it was in 1914 and again in 1916, there would be no flow into the valley. If the new levee should fail, however, the river would return to Volcano Lake and again threaten the Valley with inundation over the Volcano Lake Levee. The new levee is a substantial structure with a railroad throughout its full length and rock revetment on the river side. During the long flood season, however, the ground under and on both sides of the levee becomes saturated and softened so that, if the river in its meanders should start a direct attack, the levee might fail by undercutting in spite of all efforts to save it. Between $6 000 000 and $7 000 000 has been spent on the flood protection of Imperial Valley, of which amount the Southern Pacific Railroad Company has paid about $2 300 000 and the Federal Government, $1 100 000. (See Table 1.) The annual maintenance is said to be from $200 000 to $600 000.

A study of the reports on flood protection of the Imperial Valley shows that the engineers engaged on the work have never been alarmed over the danger of serious inundation of the Valley since the river was shut out in 1907. In this connection attention is invited to the papers by H. T. Cory,* M. Am. Soc. C. E., who closed the gap in 1907, J. C. Allison,† M. Am. Soc. C. E., and by S. L. Rothery,‡ Assoc. M. Am. Soc. C. E.

Whatever its justification, the fear of permanent inundation has grown to such an extent as to affect the ability of the Imperial District to finance and has resulted in an appeal to the Federal Government which was answered by the Act of May 18, 1920, and the "Swing-Johnson" bill. The District has contributed about $155 000 for the investigations made under the Act of 1920.

Table 1 shows the expenditures that have been made for river control below Laguna Dam.

During the past 6 years, the annual expenditures charged to maintenance§ have been as follows:

1918	$ 65 789
1919	73 471
1920	86 609
1921	122 958
1922	74 854
1923	94 835
Total	$518 516
Average	$ 86 420

* "Irrigation and River Control in the Colorado River Delta", *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

† "Control of the Colorado River as Related to the Protection of Imperial Valley", *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 297.

‡ "A River Diversion on the Delta of the Colorado in Relation to Imperial Valley, California", *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412.

§ Applies only to maintenance of levees on the Yuma Project.

TABLE 1.—Expenditures for River Control below Laguna Dam.

Reclamation Service:		
Yuma Project levees (1905–12), 54 miles:		
Earth embankment	3 040 000 cu. yd.	
Rock revetment	1 650 000 " "	
Construction and maintenance, 1905–23		$3 234 500
California Development Company:		
C. D. Co. Levee (1906–09), 27 miles, 10 miles rocked	$1 100 000	
Volcano Lake Levee (1908), 8 miles; raised and revetted in 1912; 10-mile extension, built 1914–15	525 000	
		1 625 000
U. S. Treasury—General Fund:		
Ockerson Levee (1910–11), 24.5 miles	$800 000	
Repairs and betterments to C. D. Co. and Volcano Lake Levees by Col. Ockerson (1910–11)	200 000	
Repairs to C. D. Co. and Volcano Lake Levees by Gen. Marshall (1915)	100 000	
		1 100 000
Imperial Irrigation District:		
Volcano Lake (1916–22):		
Raising and rip-rapping (approximate)	$500 000	
Fund furnished Gen. Marshall for repairs, 1915 (raised by subscription)	100 000	
Sais Levee (1919) (destroyed) (estimated)	60 000	
" " (1922) (estimated)	80 000	
Bee River Levee and Pescadero Cut-off (1921–22)	413 000	
Raising and revetting upper end of Ockerson Levee and revetment and repairs to C. D. Co. Levee (estimated)	300 000	
		1 453 000
Total cost of levees		$7 412 500
Southern Pacific Railroad:		
Closing first break, to December 1, 1906	$1 375 000	
Closing second break, December 7, 1906–July 21, 1907	1 084 000	
		$2 459 000
Total for river control		$9 871 500

Average annual cost of maintenance for past 6 years:
Yuma Project.....$ 86 500
Imperial Valley.....200 000

Expenditures on Levees, Yuma Project.

Yuma Valley Levee (built in 1905–11):		
1 125 969 cu. yd. of earthwork	$ 353 608	
26.5 miles of railroad	231 889	
Rock revetment and general maintenance	1 142 640	
		$1 728 137
Reservation Levee (built 1907–09 and 1912):		
1 166 404 cu. yd. of earthwork	$ 335 252	
Railroad	5 670	
Rock revetment and general maintenance	761 290	
		1 102 212
Gila Valley Levees:		
Arizona Levee (5½ miles built 1911):		
402 005 cu. yd. of earthwork	$ 84 996	
6.5 miles railroad	71 423	
Rock revetment and general maintenance	184 648	
		341 067
Gila Levees (7.8 miles built 1906; abandoned):		
339 804 cu. yd. of earthwork	$ 63 053	
		63 053
		$3 234 469

Totals:	
3 034 182 cu. yd. of earthwork	$ 836 909
Railroads	308 982
Rock revetment and maintenance	2 088 578
	$3 234 469

It is not possible from the data at hand to segregate the expenditures for rock revetment and maintenance prior to 1918 into amounts for current maintenance and for permanent improvements. Some of the rock has been used to slow down the erosion of natural banks at dangerous bends that were rapidly approaching the levee, and much that has been placed in concentrated quantities on the levee at temporary danger points should not be classed as permanent improvement. Of the rock, 1 650 000 cu. yd. have been placed at a cost of approximately $1 600 000, or about $1 per cu. yd. Of this amount, about three-fourths, or $1 200 000, should be considered, it is believed, as chargeable to permanent betterments. This leaves $888 578 as the current maintenance cost for the period, 1909-23, and an average of about $67 000 per year for the 13 years of active defense of the entire system.

Flood Relief Required

The difficulties in maintaining the levees in the Lower Colorado Basin have resulted in an effort to seek relief by providing storage to reduce the flood flow.

There is considerable difference of opinion as to what flow can be handled with safety in the Lower Colorado. This is not surprising because of the wide difference in flood height for any given flow. The Yuma gauge has recorded a stage of 25 ft., with flows ranging from 35 000 to 121 000 sec-ft. In a river with a stable channel, flood heights can be predicted for any given flow, but the Colorado is not such a river. It flows in a deep bed of silt and the channel enlarges as the flow increases, so that the effect of reducing the flow by storage is uncertain. The best information available on the Lower Colorado is contained in records and reports of the U. S. Bureau of Reclamation. The most accurate data regarding flow, velocity, silt content, and scour come from the Yuma gauging station and cannot be generally applied to the entire river. They do apply with fair accuracy, however, to the river from Laguna to the delta cone.

E. C. Bebb, Assoc. M. Am. Soc. C. E., has collected and compiled most of the information on which the following conclusions are based. The ability of the river to pass its flood flow without damage to the Lower Basin depends on three factors, namely, (*a*) the quantity of water that has to be passed and the rate at which it passes; (*b*) the quantity of silt carried through and deposited in the lower reaches; and (*c*) the amount of scour performed in the bed of silt through which the river flows.

The danger of breaching the levees is rarely from pressure of high water, but from undercutting due to meandering of the stream. The tendency to meander is present at all stages, but is particularly serious during a falling stage when the banks are saturated and sloughing is accelerated by reduction of hydrostatic pressure from the river. During this stage, also, meandering is accentuated because, with the checking of velocity, the heavier silt deposits rapidly in the slower water near the convex bank at bends, thus contracting the channel and deflecting the main current more and more toward the concave bank. If at this stage a small quick rise occurs that rapidly increases the velocity without giving time for the removal of the newly formed bar, the sat-

urated bank will be cut away rapidly. The records indicate that undercutting is likely to proceed more rapidly and to a greater depth the greater the quantity of water flowing in the river and, therefore, may get beyond control when the flow is very large.

The silt brought down by the river affects the flood problem in three ways:

1.—It accentuates the meandering of the stream and the instability of the channel as just described.

2.—It deposits in the delta and gradually raises the bed of the river, thereby requiring higher levees. This process is slow unless the deposit is concentrated over a small area near a levee as happened with the Volcano Lake Levee from 1909 to 1922.

3.—It may fill the flood channel excessively. Winter floods from the lower tributaries, including the Gila, carry excessive quantities of sediment, much of which deposits in the flood channel. In years when late summer and winter floods are above normal, the flood channel is filled to a point that may cause trouble when the large summer flood starts. This is shown by comparing the gauge heights at a flow of 10 000 sec-ft. (Fig. 4) with the hydrograph (Plate I).

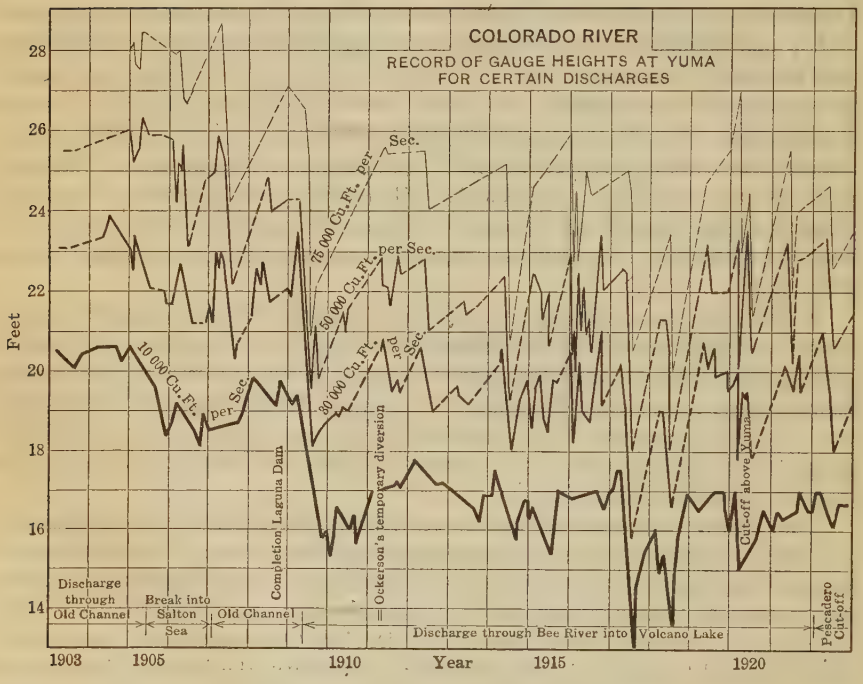
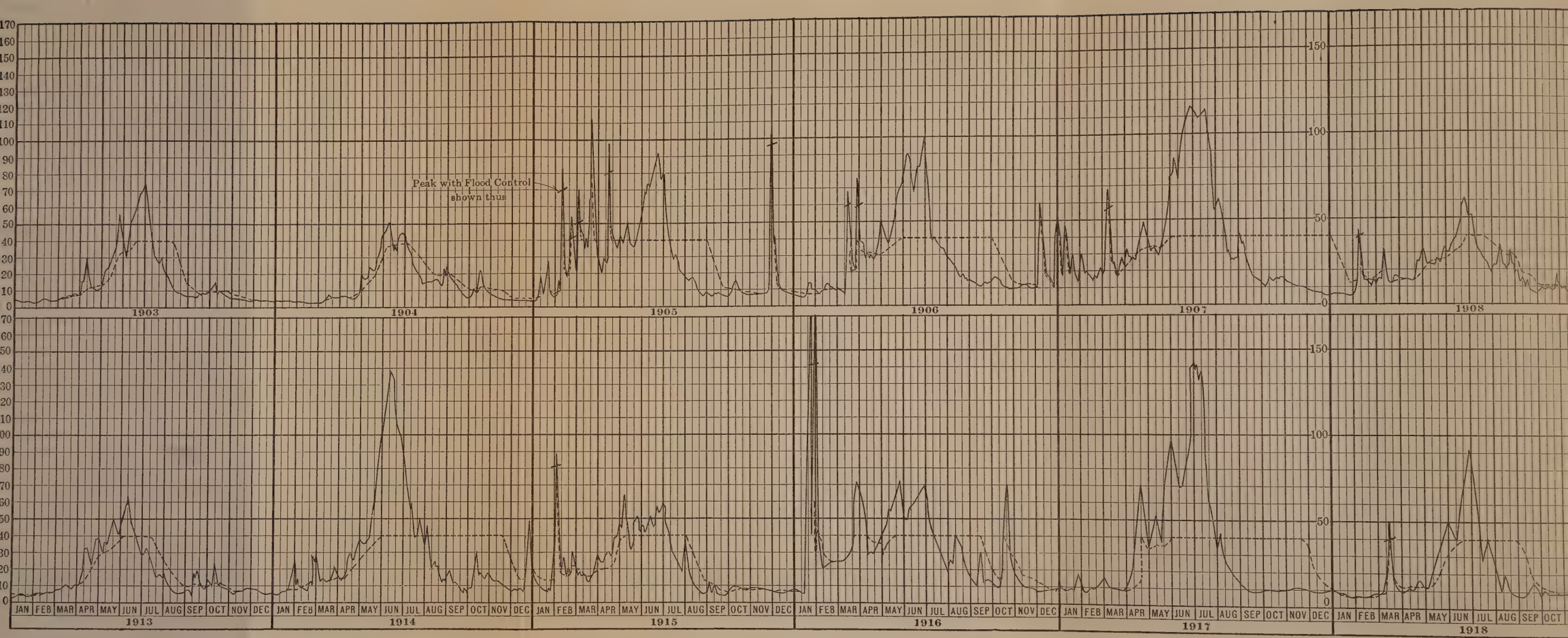


FIG. 4.

From Laguna Dam to Yuma, the river flows through a broad valley and has all the characteristics of a meandering alluvial stream. At Yuma, it is contracted by solid banks to a width of less than 600 ft. and continues in a channel


Peak with Flood Control shown thus
JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC
1903
1904
1905
1906
1907
1908
1913
1914
1915
1916
1917
1918

not more than 1 000 ft. wide for about 9 miles. This section of the river has never given serious flood trouble. The upper end of the old delta begins at this point, but in its present condition the river does not take the characteristics of a stream in its delta until it reaches the Pescadero Cut-off about 30 miles below. In the stretch from 9 to 40 miles below Yuma, the river now meanders with about the same characteristics as it does above Yuma. Just below the Pescadero Cut-off, the river spreads on its present delta cone. The area covered by the spread is relatively small and the accretions are correspondingly rapid. As the cone builds up, the delta condition probably will extend up stream so that the banks will eventually overflow at high water for some distance above the Pescadero Cut-off no matter how much the maximum flood is reduced. The normal slope of the Colorado is 1.1 ft. per mile. Under present conditions, when the Yuma gauge is below 25.4 ft., the river does not ordinarily get over its banks on the Yuma Project from Laguna to Mile 16 on the levee below Yuma, nor on the Imperial side above the junction of the Ockerson and C. D. Co. Levees. The river has passed more than 120 000 sec-ft. at this gauge height and the curves on Fig. 5 show that if the rate of increase in flow is properly retarded, the river will pass at least 80 000 sec-ft. at this stage. Below the 16-mile point, the banks are not so high above low water and the river shows a disposition to widen and not cut so deep at high water, so that the banks near the Pescadero Cut-off now overflow at a discharge of 35 000 to 50 000 sec-ft. (See Fig. 6.)

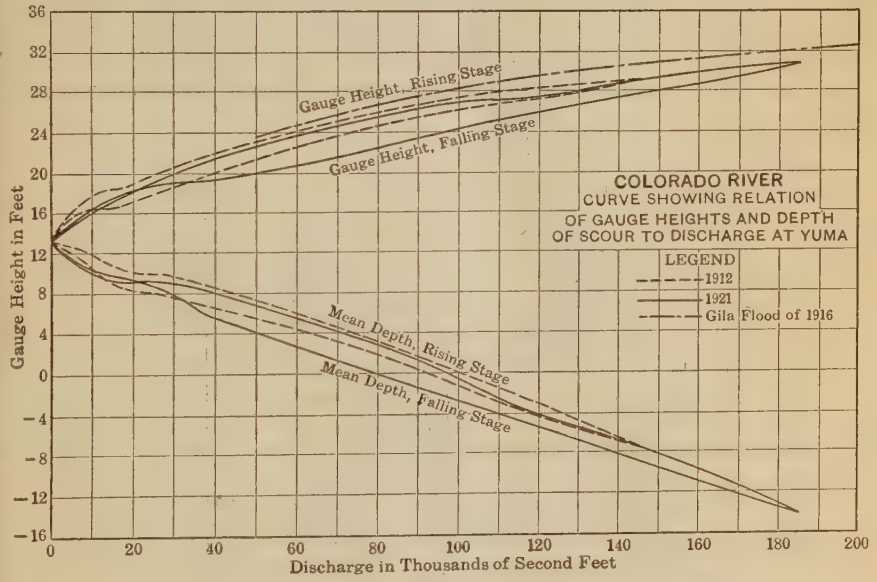


FIG. 5.

Scouring action varies theoretically as the square of the velocity. On the Colorado, active scour begins at a velocity of about 4.5 ft. per sec. and a discharge of 10 000 sec-ft. The velocity increases with discharge until a flow of about 26 000 sec-ft. is reached, at which discharge the average velocity is 6.2

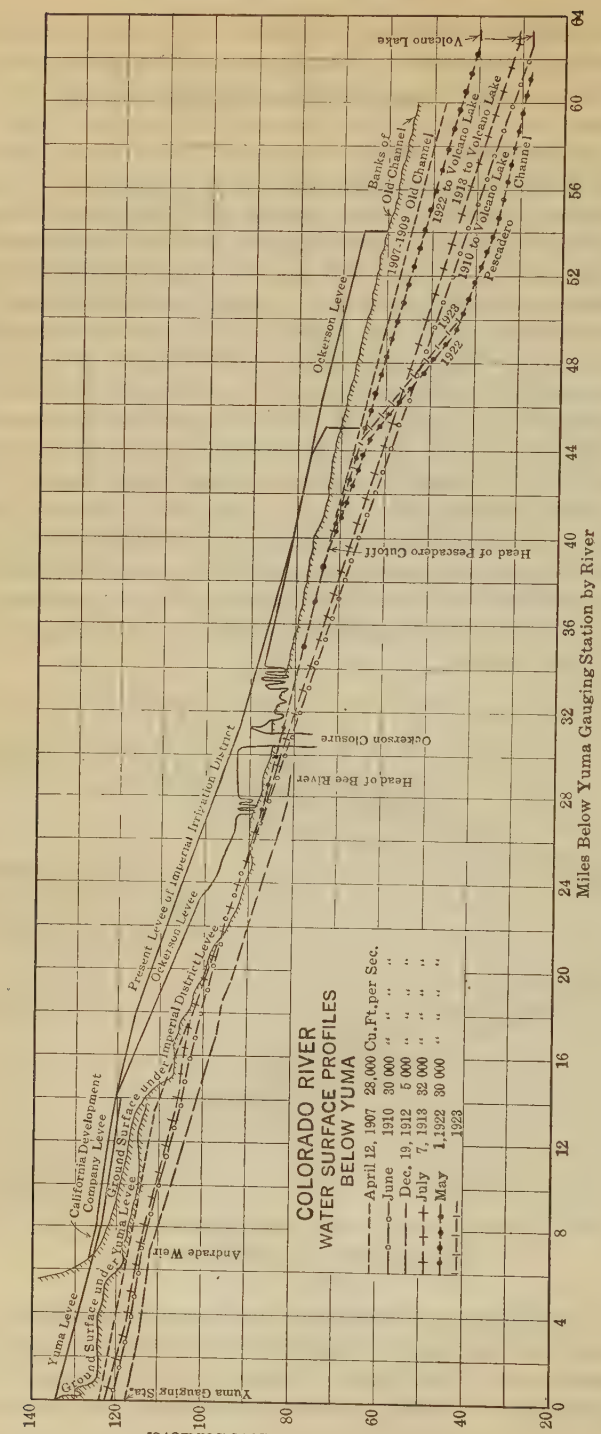

COLORADO RIVER
WATER SURFACE PROFILES
BELOW YUMA
April 12, 1907 28,000 Cu.Ft. per Sec.
June 1910 30 000 " " "
Dec. 19, 1912 5 000 " " "
July 7, 1913 32 000 " " "
May 1, 1922 30 000 " " "
1923
Elevation in Feet above Sea Level
Miles Below Yuma Gauging Station by River
Yuma Gauging Sta.
Yuma Levee
Ground Surface under Yuma Levee
California Development Company Levee
Ground Surface under Imperial District Levee
Andrade Weir
Present Levee of Imperial Irrigation District
Ockerson Levee
Ockerson Closure
Head of Bee River
Head of Pescadero Cutoff
Ockerson Levee
Banks of Old Channel 1907-1909
1922 to Volcano Lake
1918 to Volcano Lake
1910 to Volcano Lake
Pescadero Channel
1923
1922
Volcano Lake

FIG. 6.

ft. per sec. It then remains practically constant until the discharge reaches 75 000 sec-ft. after which it slowly rises. (See Fig. 7.) After a velocity of 6.2 ft. per sec. is reached, the increase in area of channel, due to scour and rise of water level, keeps pace with the increase in discharge. (See Figs. 5 and 8.) A rapid increase in discharge produces higher water levels than a slow

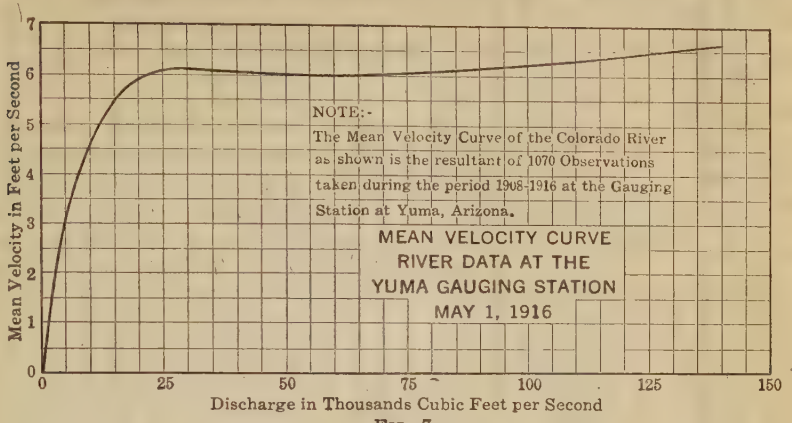


FIG. 7.

increase, because there is not time for the scour to take place. This may be seen by examination of the discharge and gauge-height curves. (See Fig. 9.) Between 20 000 and 40 000 sec-ft. appears to be a critical stage on a rising river. The silt carried within that range of discharge is excessive (see Fig. 10) and the channel on a rising river enlarges more by raising the water surface than by scour. Sudden changes in flow within that range will probably start more changes in channel than at discharges outside that range.

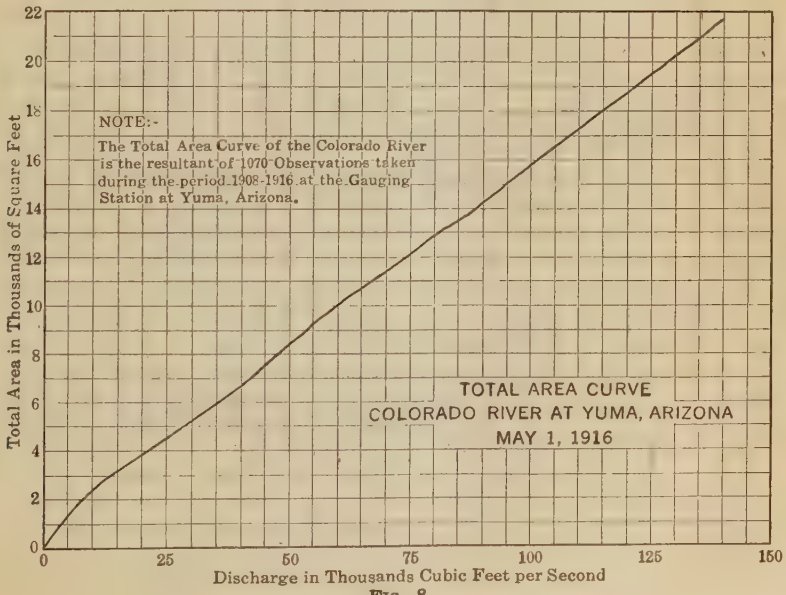


FIG. 8.

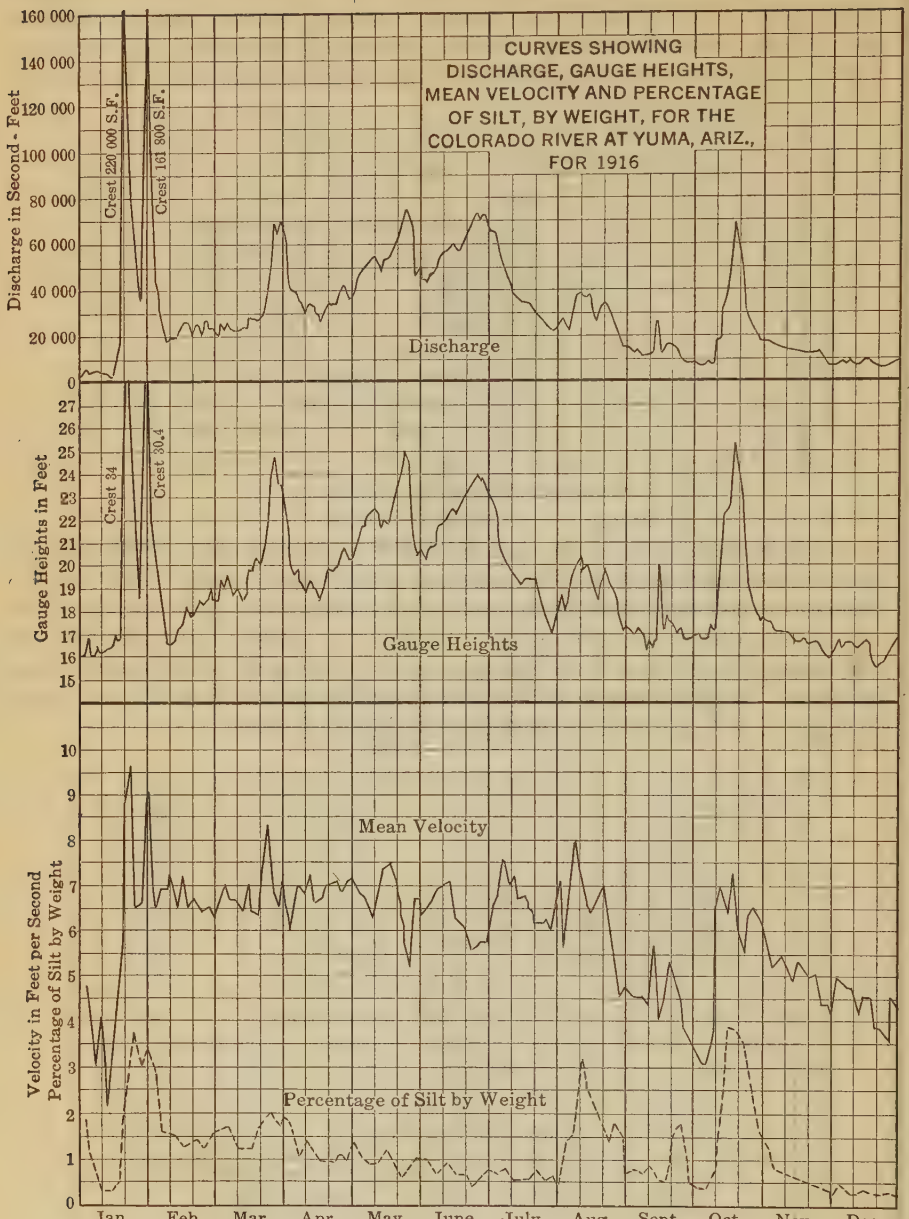

CURVES SHOWING DISCHARGE, GAUGE HEIGHTS, MEAN VELOCITY AND PERCENTAGE OF SILT, BY WEIGHT, FOR THE COLORADO RIVER AT YUMA, ARIZ., FOR 1916
Discharge in Second-Feet
160 000
140 000
120 000
100 000
80 000
60 000
40 000
20 000
0
Crest 220 000 S.F.
Crest 161 800 S.F.
Discharge
Gauge Heights in Feet
27
26
25
24
23
22
21
20
19
18
17
16
15
Crest 34
Crest 30.4
Gauge Heights
Velocity in Feet per Second
Percentage of Silt by Weight
10
9
8
7
6
5
4
3
2
1
0
Mean Velocity
Percentage of Silt by Weight
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.

FIG. 9.

If the increase in discharge from 20 000 to 40 000 sec-ft. could be made gradual and extend over a period of, say, 20 or 30 days, there probably would be less trouble due to instability of channel. After 40 000 sec-ft. is reached, the discharge can be increased more rapidly without much risk of starting new meanders. Small peaks in the flow are likely to cause trouble because they are accompanied by temporary rises in gauge heights which mean temporary rises in velocity. Examination of the operation and maintenance reports of the U. S. Bureau of Reclamation shows that such peaks are particularly objectionable on a falling river. Such peaks on a falling river have been present nearly every time the undercutting of levees has gotten beyond control. On a falling river, the discharge should be decreased slowly until the flow is about 20 000 sec-ft., in order to decrease the effect of sloughing banks and to avoid undue silting of the channel.

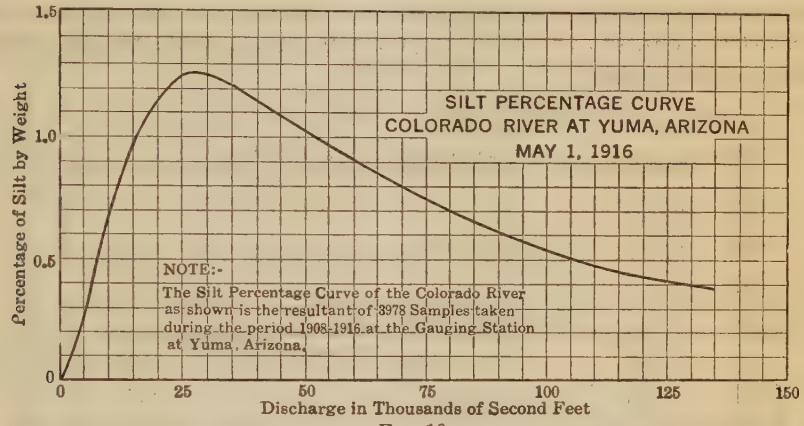


FIG. 10.

A study of the velocity curve, Fig. 7, the scour of channel curve, Fig. 5, and cross-sections on Fig. 11,* shows that the river will have the power to meander and cut its banks whenever the discharge is more than 10 000 sec-ft. It is impossible to reduce the flow to that quantity, so storage of flood water will not eliminate the necessity for bank protection and levee maintenance. The records of past flood troubles show that the river has never gone beyond control sufficiently to damage property behind the levees, except when the discharge was more than 100 000 sec-ft. If the flow is reduced to a maximum of 80 000 sec-ft. and abrupt variations in discharge are eliminated, the river will be within its banks so that danger of property damage due to flooding will be reasonably removed for the Yuma and Imperial District Projects. It is doubtful whether reducing the maximum discharge below this point will produce much saving in cost of upkeep of flood-protection works on the Yuma and Imperial District Projects and there is small chance that it will reduce the cost of property damage.

Gila floods have occurred at various times between December 1 and March 1, and they come without much warning. As any storage dam on the Colorado

* In Fig. 11, 100 ft. was added to the gauge readings to avoid the use of negative heights. In the other diagrams, straight gauge heights were used. Zero of the gauge is approximately 102 ft. above sea level.

will be 200 miles or more above Yuma, it will take at least three days for a change in discharge at the storage dam to be felt at Yuma and accurate regulation will not be possible. If the Colorado is regulated to a discharge of 40 000 sec-ft. as proposed by the U. S. Bureau of Reclamation, the flood waters will not be discharged before the middle of February in years like 1909. That means there will be a discharge of 40 000 sec-ft. in the Colorado through the period of Gila floods. It is impossible to predict what the effect of such a condition will be, but there is certainly a chance that damage from Gila floods may be increased, or that the irrigation supply may be interfered with, if an attempt is made to shut off the Colorado to conform to floods in the Gila, as proposed by the U. S. Bureau of Reclamation. It would seem wise to plan on reducing the discharge of the Colorado to the irrigation demand during the three months of probable Gila flood. In order to do this, it will be necessary for the outlet works at the dam to be capable of passing a maximum discharge in the Colorado of at least 75 000 sec-ft. so as to insure emptying the flood storage for the summer flood of the next year.

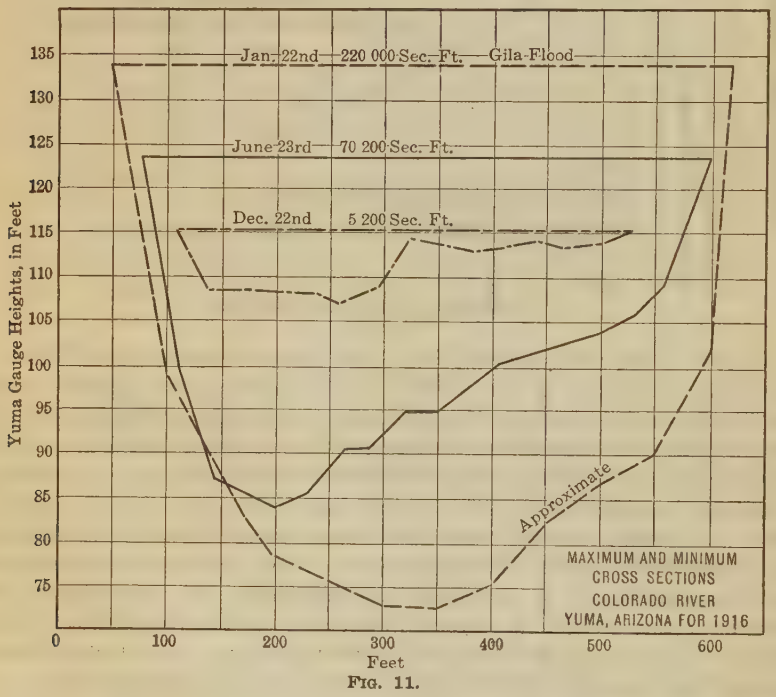


FIG. 11.

The most positive benefit that may be obtained by flood storage is that it will give insurance against permanent inundation of the Imperial Valley, provided it is great enough to permit releasing no more water than necessary to meet the irrigation demand over a period of low-water flow sufficient to enable any break into the Valley to be closed in the dry. Such a break can doubtless be closed in less than three weeks with the railroad and quarry equipment available. If storage is provided to hold the low-water flow for two months

for the year of greatest run-off on record, there will be no reason for fear of disaster to the Imperial Valley. To make this practicable a maximum discharge of 80 000 sec-ft. in the Colorado is necessary in order to free flood storage for the succeeding flood season. If the maximum discharge is fixed at 75 000 sec-ft., the same result can be obtained by adding about 600 000 acre-ft. to the flood-storage capacity.

The engineers of the U. S. Bureau of Reclamation state that, considering diversions for irrigation since 1909, the flow of 1920 may be taken to represent the largest flow of record at Yuma. They give the storage required at Boulder Canyon for flood control as shown in Table 2.

TABLE 2.

Permissible discharge at Yuma, Ariz.	Required Storage, in Millions of Acre-Feet.		
	Excess flow at Yuma.	Stream storage.	Total.
40 000 sec-ft.	6.7	0.4	7.1
50 000 sec-ft.	5.5	0.4	5.9
60 000 sec-ft.	4.4	0.3	4.7
75 000 sec-ft.	3.2	0.2	3.4

The following conclusions seem to be justified:

1.—It is not feasible, with the quantity of water now passed by the Colorado, to relieve the Lower Basin from the expense of bank protection and levee maintenance by the construction of flood storage.

2.—Flood damage to property behind the levees will probably be avoided if the maximum discharge of the Colorado is reduced to 80 000 sec-ft., and there will be little, if any, reduction in cost of bank protection and maintenance by a greater reduction in discharge.

3.—It may be desirable, until the Gila is controlled, to reduce the flow in the Colorado during the months of December, January, and February, to that required for irrigation. This means that discharge facilities with a capacity of at least 75 000 sec-ft. should be provided.

4.—It is desirable to allay the fear of permanent inundation of the Imperial Valley by making it possible to stop completely the flow below Rockwood Heading for a period sufficiently long to permit remedial works to be carried out in the dry. This can be done by using the storage provided for flood control, if flood waters are discharged at a rate of 75 000 sec-ft. and an extra 600 000 acre-ft. of storage capacity is provided.

5.—Storage for flood control is justified to a quantity that will reduce the maximum discharge for any year of record to 75 000 sec-ft. Storage in excess of that quantity may be deferred until provided as an incident to other development on the river.

6.—The conditions mentioned will be met by 4 000 000 acre-ft. of flood storage. Additional storage to allow for silting of reservoirs may be advisable. The quantity of such storage necessary will depend on the shape, location, and

method of operation of the storage reservoir and should be determined after the reservoir site is chosen.

7.—The reservoir should be designed so that when the summer flood starts, the discharge can be increased gradually without intermediate peaks. In so far as the river discharge permits, the maximum discharge of 75 000 sec-ft. should be reached by the time the June peak arrives. After the June peak has passed and the inflow to the reservoir has diminished to less than 75 000 sec-ft., the discharge should be gradually reduced, avoiding intermediate peaks until the discharge is less than 20 000 sec-ft., in order to avoid excessive sloughing and undercutting of banks and excessive silting of the channel.

8.—For flood protection of the Lower Basin, the reservoir should be as far down stream as practicable.

All-American Canal

The Swing-Johnson Bill provides that the United States Government finance and construct the so-called All-American Canal to serve the Imperial Valley and other lands in California, reimbursement to be sought from the lands benefited. The project is one that has been under consideration in one form or another for some time. It was reported on in July, 1919, by the "All-American Canal Board" and the report is printed in a public document entitled "The All-American Canal".

On October 23, 1918, an agreement was entered into between the Secretary of the Interior and the Imperial Irrigation District to provide for extension of Imperial Canal to Laguna Dam and committing the Imperial Irrigation District to the construction of an All-American Canal. This agreement was ratified by the voters of the Imperial Irrigation District on January 21, 1919, prior to the completion of the report by the All-American Canal Board, by a vote of 2 535 in favor and 922 against. Payments have been made by the Imperial District in accordance with the agreement so that the agreement is still in force but, on account of financial difficulties and the possibility that the United States might undertake the project, no construction work has been done.

Analysis of the All-American Canal project as proposed in the Swing-Johnson Bill shows that it may be considered as three separate propositions which have different degrees of urgency, as follows:

1.—Connection with Laguna Dam so that water for lands on the west side of the river may be diverted at that point instead of at the present Rockwood Heading.

2.—Serving lands in the Imperial Irrigation District through a canal entirely on the American side of the International Boundary in order to avoid complications that have arisen in the past over the operation and maintenance of a canal in Mexico.

3.—Serving lands on the East Side Mesa and in Coachella Valley which are too high to be served by the present canal system.

The proposition to connect with Laguna Dam has been under consideration for some time. It would overcome the difficulties and dangers incident to diversion at Rockwood Heading and would materially reduce the cost of

maintaining the main Imperial Canal by keeping a considerable part of the silt out of it. It would also incidentally reduce the construction costs against the Yuma Project and furnish power for its pumping area. This part of the project will cost about $6.50 per acre for the Imperial District, and the cost will be practically the same whether the connection is made to the present Imperial Canal or to the proposed All-American Canal. The Imperial District is on record as ready to pay its share of the undertaking and there is no doubt that the project should be carried out. The only question at issue is whether the Federal Government shall finance the proposition and that is a matter of policy to be determined by Congress.

The All-American Canal proposition is the result of international complications incident to operating the main Imperial Canal through Mexico. Physically, the present canal is in excellent condition. It follows the natural line for a canal and if connected to Laguna Dam and relieved of Mexican troubles would be more satisfactory than the proposed All-American Canal which must cross drifting sand hills and difficult terrain for about 15 miles in cuts more than 100 ft. deep. The cost of the All-American Canal to the Imperial District, if undertaken alone, would be about $40 per acre. If undertaken in conjunction with all other lands possible of irrigation therefrom, the cost to the Imperial District will be about $30 per acre. These costs are additional to the $6.50 per acre for connecting to Laguna Dam. The question of whether the advantages of getting out of Mexico justify the cost is one for the Imperial District to decide. It did decide in 1919, but since that time the estimate of cost has been increased and the Mexican Government has been recognized, so that the prospect of relief by treaty is improved. If the Imperial District is ready to stand by its agreement to pay for this project, as it appears to be, the only question at issue is, shall the United States finance

TABLE 3.—Cost of Connecting the Imperial Canal (or All-American Canal) with the Laguna Dam.

	Including power.	Power charged separately.
Case I.—All-American Canal as Projected. (Capacity 9 000 sec-ft. + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	$13.00	$33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	8.00	6.50
New land under All-American Canal (300 000 acres)	8.00	6.50
Case II.—Mexican Lands now Under Imperial Canal Added to All-American Canal Area. (Capacity 11 500 sec-ft. + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	13.00	33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	6.50	5.75
New lands under All-American Canal (300 000 acres)	6.50	5.75
Mexican lands under Imperial Canal (255 000 acres)	6.50	5.75
Case III.—Connection Made with Imperial Canal and Water Distributed Through That System (8 500 sec-ft + 1 600 sec-ft.):		
Yuma Project (114 000 acres)	13.00	33.00 pump lands 0.00 non-pump lands
Imperial Irrigation District (515 000 acres)	8.00	6.50
Mexican lands under Imperial Canal (255 000 acres)	8.00	6.50

TABLE 4.—Cost of All-American Canal.

(Including Cost of Connection with Laguna Dam. Yuma Project Charged for Its Interest in the Improvement.)

	Distribution system and power.	All-American Canal.	Total per acre.
Case I.—Cost if Built for New Lands Only (Excluding West Side Area) (Capacity 3 000 sec-ft.):			
(*A*). Cost of All-American Canal, not including cost of distribution systems and power installation: $18 321 000 for 265 600 acres at $69.00 per acre.			
(*B*). Cost of All-American Canal with distribution systems and power installation included:			
"A" Line Canal south of Iris (98 000 acres)	$44.70	$69.00	$113.70
"A" Line Canal north of Iris (77 100 acres)	84.00	69.00	153.00
South Side Gravity:			
(Mexico 22 000 acres: United States 8 800 acres)	20.00	69.00	89.00
"E" Line (15 700 acres)	23.50	69.00	92.50
"D" Line (Pump) (44 000 acres)	30.60	69.00	99.60
Case II.—Cost if Built for Imperial Irrigation District and New Lands Listed in Senate Document No. 142* (Capacity 9 000 sec-ft.):			
(*A*). Cost of All-American Canal, not including cost of distribution system and power installations: $29 793 000 for 813 600 acres at $36.60 per acre; $29 793 000 for 736 500 acres at $40.40 per acre. (North of Iris excluded.)			
(*B*). Cost of All-American Canal per acre with cost of distribution systems and power installations included:			
Imperial Irrigation District (515 000 acres)	4.00	36.60	40.90
"A" Line Canal south of Iris (98 000 acres)	44.70	36.60	81.30
"A" Line Canal north of Iris (77 100 acres)	84.00	36.60	120.60
South Side Gravity:			
(Mexico 22 000 acres; United States 8 800 acres.)	20.00	36.60	56.60
"E" Line (15 700 acres)	23.50	36.60	60.10
"D" Line (Pump) (44 000 acres)	30.60	36.60	67.20
"B" Line (Gravity and Pump) (38 000 acres)	56.00	36.60	92.60
Case III.—Cost if Built for Imperial Irrigation District Alone (Capacity, 6 000 sec-ft.):			
Cost of All-American Canal, without power installation or new distribution systems, but prepared for later enlargement to take care of new lands: $24 504 057†, or 515 000 acres at $47.60 per acre.			

* 67th Congress, 2d Sess.

† All-American Canal Rept., p. 96.

and build the project? Decision on this question is more or less dependent on the decision on the third proposition.

If the United States decides to undertake the irrigation of the lands too high to be reached by present canals, there will be advantages in combining with the Imperial District. The cost of an All-American Canal for the 270 000 acres that can be irrigated outside the Imperial District will be about $62.50 per acre, if undertaken alone, and $31 per acre, if undertaken in conjunction with the Imperial District. To these amounts should be added $6.50 per acre for connection with Laguna Dam and the cost of distribution systems. There is no information available to show the cost of a high canal through Mexico on a more favorable route, nor the cost of serving this land by pumping from the present canal system.

Tables 3 and 4 were prepared in order to analyze the costs of various parts of the project. They are based on unit costs of the All-American Canal Board and acreages given in the "Fall-Davis" report. It will be noted that if the

Imperial District and Mexican high lands participate in the All-American Canal, the cost of putting water on the East Mesa lands will be about $81 per acre and on Coachella Valley lands about $120 per acre. If the Imperial District and Mexican high lands do not participate in the All-American Canal, these costs will be raised to $113 and $153, respectively. On the East Side Mesa, 148 000 acres of the 160 000 irrigable are public lands. In the Coachella Valley and Dos Palmas combined, of a total of 77 000 acres irrigable, 4 500 acres are public lands and 11 400 acres are Indian lands. If the East Side Mesa and Coachella Valley lands are combined so as to average the costs, the figures become $90 per acre and $118 per acre, respectively, depending on whether or not the Imperial District participates in the All-American Canal. The data available are not sufficient to determine how much cost is justifiable to bring the 270 000 acres of lands outside Imperial District under irrigation.

Water Supply, Storage Required, and Irrigable Areas

The following is extracted from a report recently made to the Secretary of the Interior by Herman Stabler, M. Am. Soc. C. E., Chief of the Land Classification Branch of the U. S. Geological Survey:

"Water Supply of Colorado River

"The run-off of Colorado River available for future development above the diversion for the Yuma Project at Laguna Dam and for all development below this point can be expressed in terms of discharge at Laguna Dam, obtained by subtracting from the run-off of the Colorado at Yuma (*a*) the run-off of the Gila at Yuma and (*b*) depletion by reason of irrigation use and diversions from the basin above, and adding to the result thus obtained the diversions for the Yuma Project.

"In this study there have been used (*a*) records of discharge of the Colorado at Yuma since 1902; (*b*) rough estimates of discharge of the Gila at Yuma since 1903; (*c*) records of diversions for the Yuma Project since 1910; and (*d*) estimates of depletion since 1899, all furnished by the Bureau of Reclamation. In addition, gauge heights of Colorado River at Yuma from 1878 to 1922 have been used to extend the record of discharge back to 1878, and the estimates of depletion have also been extended on the assumption that irrigation in the basin began about 1850. Records of run-off of Salt River since 1889 and estimates of inflow to Great Salt Lake under the assumption of present-day development estimated from the record of Salt Lake levels have been studied as comparates, but have not been used in estimates of run-off.

"Two methods were used to develop a relation between discharge and stage at Yuma, effort being made in both to establish a method by which allowance for shifting channel conditions could be made from the record of gauge heights alone. Both methods give similar results in years of moderate flow. One method gives results rather high for years of low flow, while the other appears to give results slightly low for years of low flow. The average results by the two methods are presented as probably being more reliable than either, though apparently indicating discharges too great for years of low flow. For the period 1902-1922, in which comparison with discharge records is possible, the annual results obtained are less than the recorded discharge by as much as 10% in three years (1907, 1914, and 1920), the maximum being 20% for the year 1907. Considered as 5-year progressive means, the period 1906-1910 is found to be lower than the recorded discharge by 5%, while the period 1902-1907 is higher than the recorded discharge by 7.6%, these representing the maximum digres-

sions. These results check the accuracy of the method so far as it can be tested directly and indicate that it is fairly reliable as an indicator of mean discharge for periods of 5 years or more.

"The discharge of the Colorado as estimated for the period 1878-1922 at the diversion for the Yuma Project is given in following table [Table 5], in millions of acre-feet.

"The following table [Table 6], shows the mean discharge for various periods for Colorado River, Great Salt Lake, and Salt River, based on mean discharge for the period 1902-1922, the period of discharge records on Colorado River, all on the assumption that conditions of development were the same as in 1922 throughout the periods considered.

"The comparison here afforded tends to increase materially the confidence with which the estimated discharge prior to 1902 may be used.

"Effect of Storage on Water Supply

"Studies of water supply indicate that under conditions of development as in 1922, the water supply likely to reach Laguna Dam varies from about 6 000 000 to about 27 000 000 acre-ft. a year, and it is not unlikely that these limits may be exceeded in both directions. The mean flow (1922 conditions and equivalent flow at Laguna) for a period of 45 years, 1878-1922, is between 13 000 000 and 14 000 000 acre-ft. For periods of as much as or more than 20 years, however, the flow may be less than 11 000 000 or more than 16 000 000 acre-ft.

"TABLE 5.—Estimated Discharge of Colorado River at Laguna with 1922 Conditions of Development.
(Millions of Acre-Feet.)

Year.	Discharge.	Year.	Discharge.	Year.	Discharge.
1878	13.0	1893	11.7	1908	11.7
1879	8.7	1894	8.8	1909	24.6
1880	13.6	1895	9.4	1910	13 3
1881	11.2	1896	7.7	1911	16.8
1882	8.6	1897	10.2	1912	17.5
1883	14.0	1898	7.7	1913	11.5
1884	27.2	1899	14.0	1914	20.1
1885	15.3	1900	10.1	1915	12.6
1886	11.5	1901	14.2	1916	18.7
1887	9.1	1902	6.0	1917	19.6
1888	8.3	1903	10.3	1918	13.0
1889	9.4	1904	9.0	1919	10.3
1890	14.5	1905	14.9	1920	21.1
1891	12.9	1906	16.6	1921	19.5
1892	14.4	1907	23.9	1922	16.9
Mean	....	....	....	....	13 6

"Irrigation and power requirements both would be best served if the annual flow were made substantially uniform. The following tables [Tables 7, 8, 9, and 10], indicate what may be accomplished in this regard with specified amounts of storage capacity under the assumption that the storage reservoir or reservoirs would be full or, if of sufficient capacity, would contain 10 000 000 acre-ft. at the beginning of the 45-year period considered. The tables show the amount of water in storage each year, evaporation throughout being assumed to be 3% of the amount of water in storage. An empty reservoir and shortage in available water supply is indicated by parentheses and minus sign.

"TABLE 6.—Mean Discharge in Percentage of Mean Discharge for Period 1902-1922, with 1922 Conditions of Development.

Period.	Colorado River at Laguna.	Great Salt Lake inflow.	Salt River at Roosevelt.
1902–1922	100	100	100
1899–1922	98	97	92
1889–1922	89	90	90
1878–1922	87	89	..
1851–1922	..	92	..
1889–1901	72	78	72
1878–1901	76	78	..
1886–1904	67	75	..
1889–1904	68	75	64

"TABLE 7.—Showing Surplus or Deficiency of Water Supply with Regulation in a Reservoir of 5 000 000 Acre-Ft. Capacity for Specified Annual Demand.

(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand:	
		10.	11.
1877	....	5.0	5.0
1878	13.0	5.0	5.0
1879	8.7	3.5	2.5
1880	13.6	5.0	5.0
1881	11.2	5.0	5.0
1882	8.6	3.5	2.5
1883	14.0	5.0	5.0
1884	27.2	5.0	5.0
1885	15.3	5.0	5.0
1886	11.5	5.0	5.0
1887	9.1	3.9	2.9
1888	8.3	2.1	0.1
1889	9.4	1.5	(—1.5)
1890	14.5	5.0	3.5
1891	12.9	5.0	5.0
1892	14.4	5.0	5.0
1893	11.7	5.0	5.0
1894	8.8	3.6	2.6
1895	9.4	2.9	0.9
1896	7.7	0.5	(—2.4)
1897	10.2	0.7	(—0.8)
1898	7.7	(—0.7)	(—3.3)
1899	14.0	4.0	3.0
1900	10.1	4.0	2.0
1901	14.2	5.0	5.0
1902	6.0	0.8	(—0.2)
1903	10.3	1.1	(—0.7)
1904	9.0	0.0	(—2.0)
1905	14.9	4.9	3.9
1906	16.6	5.0	5.0
1907	13.9	5.0	5.0
1908	11.7	5.0	5.0
1909	24.6	5.0	5.0
1910	13.3	5.0	5.0
1911	16.8	5.0	5.0
1912	17.5	5.0	5.0
1913	11.5	5.0	5.0
1914	20.1	5.0	5.0
1915	12.6	5.0	5.0
1916	18.7	5.0	5.0
1917	19.6	5.0	5.0
1918	13.0	5.0	5.0
1919	10.3	5.0	4.1
1920	21.1	5.0	5.0
1921	19.5	5.0	5.0
1922	16.9	5.0	5.0

"TABLE 8.—Showing Surplus or Deficiency of Water Supply with Regulation in a Reservoir of 10 000 000 Acre-Ft. Capacity for Specified Annual Demand.

(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand:					
		10.	11.	11.5.	12.	12.5.	13.
1877	....	10.0	10.0	10.0	10.0	10.0	10.0
1878	13.0	10.0	10.0	10.0	10.0	10.0	9.7
1879	8.7	8.7	7.7	6.9	6.4	5.9	5.1
1880	13.6	10.0	10.0	8.8	7.8	6.8	5.5
1881	11.2	10.0	9.9	8.2	6.8	5.3	3.5
1882	8.6	8.6	7.2	5.1	3.2	1.2	(−1.0)
1883	14.0	10.0	10.0	7.4	5.1	2.7	1.0
1884	27.2	10.0	10.0	10.0	10.0	10.0	10.0
1885	15.3	10.0	10.0	10.0	10.0	10.0	10.0
1886	11.5	10.0	10.0	9.7	9.2	8.7	8.2
1887	9.1	9.1	7 8	7.0	6.0	5.0	4.1
1888	8.3	7.1	4.8	3.6	2.1	0.6	(—0.7)
1889	9.4	6.3	3.0	1.4	(−0.6)	(—2.1)	(—3.6)
1890	14.5	10.0	6.4	4.4	2.5	2.0	1.5
1891	12.9	10.0	8.1	5.7	3.3	2.3	1.4
1892	14.4	10.0	10.0	8.4	5.6	4.1	2.8
1893	11.7	10.0	10.0	8.4	5.1	3.2	1.4
1894	8.8	8.8	7.5	5.4	1.7	(−0.6)	(—2.8)
1895	9.4	7.9	5.7	3.1	(—1.0)	(—3.1)	(—3.6)
1896	7.7	5.4	2.2	(—0.8)	(—4.3)	(—4.8)	(—5.3)
1897	10.2	5.4	1.3	(—1.3)	(—1.8)	(—2.3)	(—2.8)
1898	7.7	3.0	(—2.0)	(—3.8)	(—4.3)	(—4.8)	(—5.3)
1899	14.0	6.9	3.0	2.5	2.0	1.5	1.0
1900	10.1	6.8	2.0	1.5	0.0	(—0.9)	(—1.9)
1901	14.2	10.0	5.1	4.2	2.2	1.7	1.2
1902	6.0	6.0	0.0	(—0.6)	(—3.9)	(—4.8)	(—5.8)
1903	10.3	6.1	(—0.7)	(—1.2)	(—1.7)	(—2.2)	(—2 7)
1904	9.0	4.9	(—2.0)	(—2.5)	(—3.0)	(—3.5)	(—4.0)
1905	14.9	9.7	3.9	3.4	2.9	2.4	1.9
1906	16.6	10.0	9.4	8.4	7.4	6.4	5.4
1907	13.9	10.0	10.0	10.0	9.1	7.6	6.1
1908	11.7	10.0	10.0	9.9	8.5	6.6	4.6
1909	24.6	10.0	10.0	10.0	10.0	10.0	10.0
1910	13.3	10.0	10.0	10.0	10.0	10.0	10.0
1911	16.8	10.0	10.0	10.0	10.0	10.0	10.0
1912	17.5	10.0	10.0	10.0	10.0	10.0	10.0
1913	11.5	10.0	10.0	9.7	9.2	8.7	8.2
1914	20.1	10.0	10.0	10.0	10.0	10.0	10.0
1915	12.6	10.0	10.0	10.0	10.0	10.8	9.3
1916	18.7	10.0	10.0	10.0	10.0	10.0	10.0
1917	19.6	10.0	10.0	10.0	10.0	10.0	10.0
1918	13.0	10.0	10.0	10.0	10.0	10.0	9.7
1919	10.3	10.0	9.0	8.5	8.0	7.5	6.7
1920	21.1	10.0	10.0	10.0	10.0	10.0	10.0
1921	19.5	10.0	10.0	10.0	10.0	10.0	10.0
1922	16.9	10.0	10.0	10.0	10.0	10.0	10.0

"TABLE 9.—Showing Surplus or Deficiency in Water Supply with Regulation in a Reservoir of 20 000 000 Acre-Ft. Capacity for Specified Annual Demand.
(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand:			
		11.	12.	13.	14.
1877	....	10.0	10.0	10.0	10.0
1878	13.0	11.7	10.7	9.7	8.7
1879	8.7	9.1	7.1	5.1	3.1
1880	13.6	11.4	7.8	5.5	2.6
1881	11.2	11.3	6.8	3.5	(—0.3)
1882	8.6	8.6	3.2	(—1.0)	(—5.4)
1883	14.0	11.3	5.1	1.0	(—0.0)
1884	27.2	20.0	20.0	15.2	13.2
1885	15.3	20.0	20.0	17.1	14.1
1886	11.5	19.9	18.9	15.1	11.2
1887	9.1	17.4	15.4	10.8	6.0
1888	8.3	14.2	11.2	5.8	0.1
1889	9.4	12.2	8.3	2.0	(—4.5)
1890	14.5	15.3	10.6	3.4	0.5
1891	12.9	16.7	10.7	3.2	(—0.6)
1892	14.4	19.6	12.8	4.5	0.4
1893	11.7	19.7	12.1	3.1	(—1.9)
1894	8.8	16.9	8.5	(—1.2)	(—5.2)
1895	9.4	14.8	5.6	(—3.6)	(—4.6)
1896	7.7	11.1	1.1	(—5.3)	(—6.3)
1897	10.2	10.0	(—0.7)	(—2.8)	(—3.8)
1898	7.7	6.4	(—4.3)	(—5.3)	(—6.3)
1899	14.0	9.2	2.0	1.0	0.0
1900	10.1	8.0	0.0	(—1.9)	(—3.9)
1901	14.2	11.0	2.2	1.2	0.2
1902	6.0	5.7	(—3.9)	(—5.8)	(—7.8)
1903	10.3	4.8	(—1.7)	(—2.7)	(—3.7)
1904	9.0	2.6	(—3.0)	(—4.0)	(—5.0)
1905	14.9	6.4	2.9	1.9	0.9
1906	16.6	11.8	7.4	5.4	3.5
1907	13.9	14.3	9.1	6.1	3.3
1908	11.7	14.5	8.5	4.6	0.9
1909	24.6	20.0	20.0	16.1	11.5
1910	13.3	20.0	20.0	15.9	10.5
1911	16.8	20.0	20.0	19.2	13.0
1912	17.5	20.0	20.0	20.0	16.1
1913	11.5	19.9	18.9	17.9	13.1
1914	20.1	20.0	20.0	20.0	18.7
1915	12.6	20.0	20.0	19.0	16.7
1916	18.7	20.0	20.0	20.0	20.0
1917	19.6	20.0	20.0	20.0	20.0
1918	13.0	20.0	20.0	19.4	18.4
1919	10.3	18.7	17.7	16.3	14.1
1920	21.1	20.0	20.0	20.0	20.0
1921	19.5	20.0	20.0	20.0	20.0
1922	16.9	20.0	20.0	20.0	20.0

"TABLE 10.—Showing Surplus or Deficiency in Water Supply with Regulation in a Reservoir of 30 000 000 Acre-Ft. Capacity for Specified Annual Demand.

(Millions of Acre-Feet.)

Years.	Estimated flow at Laguna.	Annual Demand.		
		12.	13.	14.
1877	....	10.0	10.0	10.0
1878	13.0	10.7	9.7	8.7
1879	8.7	7.1	5.1	3.1
1880	13.6	7.8	5.5	2.6
1881	11.2	6.8	3.5	(—0.3)
1882	8.6	3.2	(—1.0)	(—5.4)
1883	14.0	5.1	1.0	(—0.0)
1884	27.2	20.1	15.2	13.2
1885	15.3	22.8	17.1	14.1
1886	11.5	21.6	15.1	11.2
1887	9.1	18.1	10.8	6.0
1888	8.3	13.9	5.8	0.1
1889	9.4	10.9	2.0	(—4.5)
1890	12.9	13.6	3.2	(—0.6)
1891	14.5	13.1	3.4	0.5
1892	14.4	15.6	4.5	0.4
1893	11.7	14.8	3.1	(—1.9)
1894	8.8	11.2	(—1.2)	(—5.2)
1895	9.4	8.3	(—3.6)	(—4 6)
1896	7.7	3.8	(—5.3)	(—6.3)
1897	10.2	1.9	(—2.8)	(—3.8)
1898	7.7	(—2.5)	(—5.3)	(—6.3)
1899	14.0	2.0	1.0	0.0
1900	10.1	0.0	(—1.9)	(—3.9)
1901	14.2	2.2	1.2	0.2
1902	6.0	(—3.9)	(—5.8)	(—7.8)
1003	10.3	(—1.7)	(—2.7)	(—3.7)
1904	9.0	(—3.0)	(—4.0)	(—5.0)
1905	14.9	2.9	1.9	0.9
1906	16.6	7.4	5.4	3.5
1907	13.9	9.1	6.1	3.3
1908	11.7	8.5	4.6	0.9
1909	24.6	20.8	16.1	11.5
1910	13.3	21.5	15.9	10.5
1911	16.8	25.7	19.2	13.0
1912	17.5	30.0	23.1	16.1
1913	11.5	28.6	20.9	13.1
1914	20.1	30.0	27.4	18.7
1915	12.6	29.7	26.2	16.7
1916	18.7	30.0	30.0	20.9
1917	19.6	30.0	30.0	25.9
1918	13.0	30.0	29.1	24.1
1919	10.3	27.4	25.5	19.7
1920	21.1	30.0	30.0	26.2
1921	19.5	30.0	30.0	30.0
1922	16.9	30.0	30.0	30.0

"In the foregoing summary the percentage of maximum shortage is based on the stated amount of annual demand. As irrigation development proceeds, the annual flow at the storage reservoirs would be decreased by more than 1 000 000 acre-ft. when the area of lands estimated to be irrigable in the immediate future receives water, by over 2 000 000 acre-ft. when the area estimated to be irrigable in the near future receives water, and by about 5 000 000 acre-ft. when the area estimated to be irrigable in the distant future receives water. Furthermore, it cannot reasonably be assumed that the shortage will be prorated fully over the lands below Boulder Canyon that now receive a full water supply and require 3 000 000 to 4 000 000 acre-ft. for irrigation. The water shortages for new lands below Boulder Canyon will therefore be greatly increased over the percentages indicated in Table [11]. If irrigation development below Boulder Canyon is based on the theory that a flow of 14 000 000 acre-ft., or even of 13 000 000 acre-ft., can be maintained annually, years are indicated for the distant future when not only would there be no water for new lands, but lands now irrigated would receive a scant supply regardless of whether the storage capacity available be 10 000 000, 20 000 000, or 30 000 000 acre-ft. The data indicate that it is unsafe at this time to plan development on the basis of an annual flow of more than between 11 000 000 and 12 000 000 acre-ft.

"The data in Tables 7, 8, 9, and 10 are summarized, as follows:

	Storage Capacity, in Millions of Acre-Feet.			
	5.	10.	20.	30.
Annual Demand of 10 000 000 Acre-Feet:				
Shortage, percentage of time	2	0	....	....
Maximum shortage, percentage	7	0	....	....
Overflow, percentage of time	64	62	....	....
Annual Demand of 11 000 000 Acre-Feet:				
Shortage, percentage of time	16	7	0	....
Maximum shortage, percentage	30	Under 20	0	....
Overflow, percentage of time	60	49	31	....
Annual Demand of 11 500 000 Acre-Feet:				
Shortage, percentage of time	....	13	....	....
Maximum shortage, percentage	....	33	....	....
Overflow, percentage of time	....	33	....	....
Annual Demand of 12 000 000 Acre-Feet:				
Shortage, percentage of time	....	18	11	9
Maximum shortage, percentage	....	36	36	32
Overflow, percentage of time	....	28	31	16
Annual Demand of 12 500 000 Acre-Feet:				
Shortage, percentage of time	....	22	....	....
Maximum shortage, percentage	....	38	....	....
Overflow, percentage of time	....	26	....	....
Annual Demand of 13 000 000 Acre-Feet:				
Shortage, percentage of time	....	27	22	22
Maximum shortage, percentage	....	45	41	41
Overflow, percentage of time	....	22	16	11
Annual Demand of 14 000 000 Acre-Feet:				
Shortage, percentage of time	....	....	31	31
Maximum shortage, percentage	....	....	55	55
Overflow, percentage of time	....	....	11	4

"The storage capacity adequate to maintain reasonably uniform annual flow is difficult to determine. It is evidently greater than 5 000 000 and pretty certainly as great as 10 000 000 acre-ft. and less than 20 000 000 acre-ft. Capacity of 10 000 000 acre-ft. will provide annual flow of 11 500 000 acre-ft. with somewhat less onerous conditions of shortage than if capacity of 20 000 000 acre-ft. were utilized to provide annual flow of 12 000 000 acre-ft.

"Increase of Storage Capacity from 10 000 000 Acre-Ft. to the Available Annual Water Supply.

"According to estimates of Bureau of Reclamation engineers the cheapest known storage capacity of 10 000 000 acre-ft. would be the top 65 ft. (approximately) of the large Boulder Canyon reservoir and would cost about $10 000 000. At $1 per acre-foot of storage capacity the increase in storage capacity from 10 000 000 to 20 000 000 acre-ft. would cost more than $20 per acre-foot of increase in mean annual flow. This would be a doubtful investment if the increase in possible power and irrigation development at and below Boulder Canyon be alone considered, but may be justifiable if the additional storage capacity be provided up stream so that the additional flow could be utilized for power purposes through the much greater head available, in the canyon section of Colorado River.

"It is concluded that storage capacity in excess of 10 000 000 acre-ft. should not be provided, if at all, until the conditions of water supply under future development are much better known than at present.

"Effect of Estimates of Water Supply and Practicable Storage Control on Possibilities of Irrigation Development

"The estimates of water supply and practicable storage for the period 1878-1922 indicate that through long periods not to exceed 12 000 000 acre-ft. of water a year may be relied on for future irrigation development above Laguna Dam and for present and future development below that point.

"Compilation of irrigable areas in the basin of Colorado River and the adjacent basin of Salton Sea, classified by degree of feasibility, has been made by engineers of the Bureau of Reclamation. These areas and estimated net consumption of water by them and by probable diversions from the basin are set forth in the following table [Table 11]:

"TABLE 11.—Irrigable Areas and Water Requirements.

(Areas in Thousands of Acres and Shown to Nearest 10 000. Water Consumption in Thousands of Acre-Feet and Shown to Nearest 100 000.)

	Present:		Immediate Future:		Near Future:		Distant Future:		Total:	
	Area.	Water.	Area.	Water.	Area.	Water.	Area.	Water.	Area.	Water.
Above Boulder.....	....	....	770	1 300	1 060	2 000	970	1 900	2 900	5 200
Between Boulder and Laguna.....	....	....	170	500	50	200	870*	3 900	1 090	4 600
Below Laguna in United States....	470	2 100	430	1 900	....	....	....	....	900	4 000
Total in United States.....	470	2 100	1 370	3 700	1 110	2 100	1 840	5 800	4 990	13 600
In Mexico.....	190	900	300	1 300	310	1 400	200†	900	1 000	4 500
Grand total....	660	3 000	1 700	5 000	1 410	3 500	2 040	6 600	5 810	18 100

"* Includes a little over 800 000 acres listed in available areas by Bureau of Reclamation engineers, but excluded from their estimates of water consumption with the following statement: The Parker-Gila Valley Project is not considered fully as feasible as other Class C projects in either basin and as available water supply is insufficient for the inclusion of its entire area at the assumed diversion duty, the area of this project is reduced to fit the water supply.

"† This area not dependent on flow of Colorado River at Laguna as it may be served by return flow, waste, etc., near head of Gulf.

"The foregoing table [Table 11] does not include 1 500 000 acres of land now irrigated above Laguna Dam, requirements of which have been provided for and excluded from the estimates of water supply.

"The conclusion is obvious that the water supply is inadequate to provide for a very large area in the projects that have been classified as becoming feasible only in the distant future. However, before such projects are seriously considered for development, far more and better information as to water supply will be available and a reasonably accurate estimate of the adequacy or inadequacy of the water supply available for them can be made. An attempt to solve all the problems of Colorado River at a single stroke on the basis of the meager information now available is likely to result in ill-advised expenditure if nothing worse."

Storage Required at Present for Irrigation of Lower Basin

The entire low-water flow of the Colorado in the Lower Basin is now appropriated, and storage will be necessary for any expansion of acreage and a small amount of storage should be provided to take care of present acreage in years of less than average water supply. The Fall-Davis report (page 40) shows that, in the year of smallest run-off, 2 340 000 acre-ft. of storage would be sufficient to provide for the feasible acreage in both the Upper and the Lower Basins. Since the Fall-Davis report was issued, the U. S. Bureau of Reclamation has made extensive studies of possible irrigation in the Colorado Basin, the net results of which indicate that all lands likely to be irrigated by 1940 in both the Upper and the Lower Basins can be provided for by 2 300 000 acre-ft. of storage in the worst year of record. Aside from 1902, which was the worst year, 1 500 000 acre-ft. of storage would have been sufficient.

About 840 000 acre-ft. of storage will take care of all development contemplated in the Swing-Johnson Bill, including that in Mexico supplied by the Imperial Canal, for years like 1902, and 500 000 acre-ft. would be sufficient for all other years of record. If 4 000 000 acre-ft. of storage is provided for flood control, it can be operated in such manner as to take care of irrigation needs in the Lower Basin as far as it is profitable to do so at this time.

Power

The power demand in Southern California has increased in the past 14 years at a rate of about 13% compounded annually. There is a present mining load in Arizona carried by steam power that justifies the belief that 75 000 to 80 000 kw. could be absorbed from the Colorado as soon as it is available, and the development of power on the Colorado at Diamond Creek would now be under way but for lack of Federal authority to use the public lands necessary. There are mining possibilities in Nevada and Southern Utah that may create a large demand. They are too speculative at present to justify expenditure for power development, but may greatly accelerate such development once it is started. The mining load in the vicinity of Salt Lake City has grown so that development of the Flaming Gorge site on the Green River, just below the Wyoming line, is now under serious consideration by the Utah Power and Light Company.

The use of power in the United States has grown with surprising rapidity. When it is considered that the use of electrical power commenced about 30 years ago, it is not extravagant to conclude that all the power in the Canyon Section of the Colorado may be absorbed in the next 25 or 30 years. Although immediate expenditure should be limited to immediate needs, the possibility

of such a demand for power is certainly great enough to justify taking it into account in planning the development of the Colorado, to the extent of insuring the ultimate development of the greatest possible power from the Canyon Section commensurate with the higher uses of the water. This means that projects should be planned to use all available head, that storage for equalization of flow should be provided above the Canyon Section, and that storage for re-regulation of flow for irrigation in the Lower Basin should be provided at the bottom of the Canyon Section. Storage located at the bottom of the Canyon Section for flood protection of the Lower Basin does not conform to the requirement that storage for equalization of flow should be above the Canyon Section. It should be built, therefore, only to the minimum capacity that will give relief, especially as storage above the canyon may be relied on to give additional relief in the near future, and it should be located, if practicable, so that later it will provide for re-regulation of flow for irrigation in the Lower Basin.

Aside from the Flaming Gorge and Diamond Creek developments on the Colorado, any developments of power in the immediate future must depend on Southern California for a market, and the size of the development should be limited to what that market can absorb with certainty, in order to keep the carrying charges at a minimum during the periods of construction and loading.

Table 12 has been compiled from information in the files of the Federal Power Commission to show the possible hydro-electric development in Southern California.

The projected developments of the San Joaquin Light and Power Company are on the North Fork of Kings River. Expenditures on these works have been commenced on a scale that will probably make necessary their completion in spite of the fact that their estimated cost is more than $200 per h.p. installed.

The projected developments of the Southern California Edison Company will probably go forward up to 1935. The installed capacity for that year includes what are known as the West Side Developments of the San Joaquin River. They will be expensive, costing probably more than $200 per h.p. installed, and may be deferred if Colorado River power becomes available.

The proposed developments of the City of Los Angeles are in two regions, namely, Owens Valley and South Fork of Kings River. The developments in the latter region, at least, may be deferred if Colorado power becomes available.

Table 13 shows the power generated in Southern California by the four principal producers. Their combined load has grown at an average rate of about 13% compounded annually. Nearly 80% of the power has been hydropower and has operated at an annual capacity factor of from 45 to 55 per cent.

Table 14 shows predictions of growth of load in Southern California. The prediction* made by Frederick H. Fowler, M. Am. Soc. C. E., is stated by him to be the probable maximum. The prediction† made by A. H. Mark-

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 836.

† *General Electric Review*, 1922.

TABLE 12.—California Hydro-Power Projects.

Companies.	Streams.	September 1921, installed capacities, in kilovolt-amperes.	Program of Development, in Kilovolt-Amperes: 1922	1923	1924	1925	1926	1927	1928	1929	1930	1935	1940	1945	Ultimate capacity of new projects.
San Joaquin	Kings River No. 175	82 450	.....	.....	.....	.....	15 000	15 000	.....	15 000	15 000	30 000	60 000	147 000	297 000
So. Calif. Edison	Big Creek No. 67	201 350	30 000	16 000	.....	25 000	.....	.....	.....	.....	.....	.....	.....	.....	71 000
"	San Joaquin	.....	.....	75 000	.....	16 000	.....	99 000	99 000	.....	195 000	212 000	.....	.....	696 000
Nev.-Calif. Power	Leevining, Rush, Owens, and Snow Creeks	40 100	.....	2 800	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 800
So. Sierras		.....	.....	.....	.....	3 000	7 500	.....	.....	.....	12 500	.....	.....	.....	23 000
City of Los Angeles		72 460	.....	.....	.....	2 000	1 500	.....	.....	.....	.....	.....	.....	.....	3 500
San Gorgonio	San Gorgonio	.....	.....	.....	2 700	.....	.....	.....	.....	.....	.....	.....	.....	.....	654 750
Sespe Light and Power	Piru Creek	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	2 700
Merced Irrig.	Merced River No. 88	.....	.....	.....	.....	.....	6 000	.....	.....	.....	7 000	.....	.....	.....	13 000
Fletcher, Edison	Boulder Creek	.....	.....	.....	.....	30 000	.....	.....	.....	.....	.....	.....	.....	.....	30 000
		.....	.....	.....	.....	.....	.....	.....	.....	.....	980	.....	.....	.....	980
Total	.....	396 360	30 000	93 800	2 700	76 000	30 000	114 000	116 500	52 500	263 930	288 080	186 220	541 000	1 794 730
Accumulative total	.....	.....	.....	123 800	126 500	202 500	232 500	346 500	463 000	515 500	779 430	1 067 510	1 253 730	1 794 730	.....
		.....	426 360	520 160	522 860	598 860	628 860	742 860	859 360	911 860	1 175 790	1 463 870	1 650 090	2 191 090	.....

wart, M. Am. Soc. C. E., was for the entire State of California and is probably low for Southern California. Column (3) of Table 14 which was compiled from Mr. Fowler's prediction on the assumption that hydro-developments will continue to furnish 80% of the power and will operate at an annual capacity factor of 55%, would be high as a basis for immediate investment if it were not for the fact, as mentioned previously, that some of the California installation may be deferred in favor of Colorado development. Column (5) of Table 14 contains the totals from Table 12. A comparison with Column (3) shows that California developments will probably take care of the growth of load in Southern California up to 1935. At that time the growth of load will call for increased installation of about 100 000 kw. per year.

TABLE 13.—Power Generated in Southern California.*

(Systems Included: Southern California Edison Company; Southern Sierras Power Company; San Joaquin Light and Power Corporation; and City of Los Angeles.)

Year.	Hydro:		Steam:		Total:		Percentage hydro.
	Kilowatt-hours.	Percentage increase.	Kilowatt-hours.	Percentage increase.	Kilowatt-hours.	Percentage increase.	
1910	324 331 916	....	124 181 024	....	448 512 940	....	72.3
1911	376 988 076	16.2	162 675 931	30.6	539 664 007	20.33	69.8
1912	383 136 890	1.6	273 354 263	68.0	656 491 153	21.64	58.35
1913	431 519 017	12.6	307 293 993	12.45	738 804 010	12.53	58.40
1914	740 991 812	71.7	104 288 601	—66.26	845 780 413	14.47	87.61
1915	788 344 373	6.4	93 557 059	—10.29	881 901 432	4.10	89.39
1916	876 636 452	11.11	63 578 530	—32.05	940 214 982	65.70	93.23
1917	924 781 935	5.4	165 909 200	160.80	1 090 691 135	16.00	84.78
1918	1 005 383 515	8.7	250 108 913	50.75	1 255 492 428	15.10	80.07
1919	986 852 219	—1.8	414 084 672	65.56	1 400 936 891	11.58	70.44
1920	1 161 522 861	1.76	432 753 084	4.50	1 594 275 945	13.80	72.85
1921	1 514 873 779	30.5	314 532 165	—27.32	1 829 405 944	14.74	82.08
1922	1 848 494 288	22.0	123 615 130	—60.7	1 972 109 418	7.80	93.73
1923	1 992 877 000	7.8	406 469 000	22.88	2 399 346 000	21.66	83.05
Total..	12 356 725 133	15.0†	3 236 401 698	9.5†	15 593 126 698	13.8†	79.8

* Compiled from *Water Supply Paper No. 498*, U. S. Geological Survey, and records of U. S. Geological Survey.

† Compound rate of increase for entire period.

A Colorado River power development probably cannot be completed before 1930, but should be completed by 1932. In view of the statements made, the estimates of power company engineers that a Colorado River development planned now should not exceed an installed capacity of 300 000 kw. appears to be sound. A larger development will ultimately be absorbed, but the risk of excessive carrying charges is not justifiable if the smaller development is feasible. It has become the custom to talk in such large figures concerning the Colorado that it may be pertinent to point out that the only hydro-electric developments in this country as large as 300 000 kw. are at Niagara Falls and Muscle Shoals, and that the total installed capacity in Southern California at present is about 500 000 kw.

TABLE 14.—Probable Power Demand of Southern California.

Year.	Per Fowler: Increase at: 13% to 1925 12% to 1930 10% to 1935 8% to 1940 in millions of kilowatt-hours. (1)	Per Markwart: Increase at: 11.1% to 1930 7.2% to 1940 4.1% to 1950 in millions of kilowatt-hours. (2)	Fowler's Estimate: Assume hydro produces 80% of kilowatt-hours and operates at 55% capacity factor. Required installed capacity, in kilowatts. (3)	Markwart's Estimate: Hydro produces 90% of kilowatt-hours and operates at 72% capacity factor. Required installed capacity, in kilowatts. (4)	Possible installed capacity of Southern California hydro per Table 12. (5)
1924	2 710	2 662	451 660	384 400	522 860
1925	3 062	2 954	510 330	435 700	598 860
1926	3 429	3 280	571 490	483 900	628 860
1927	3 840	3 640	640 000	537 000	742 860
1928	4 301	4 040	716 820	596 000	859 360
1929	4 817	4 484	802 820	661 600	911 860
1930	5 395	4 975	899 150	733 900	1 175 790
1931	5 935	5 109	991 510	753 700	.....
1932	6 528	5 477	1 088 000	808 000	.....
1933	7 181	5 879	1 196 810	867 700	.....
1934	7 899	6 302	1 316 470	930 000	.....
1935	8 689	6 755	1 448 140	996 700	1 463 870
1936	9 384	7 241	1 563 970	1 068 000	.....
1937	10 134	7 762	1 689 000	1 145 000	.....
1938	10 944	8 369	1 823 970	1 234 000	.....
1939	11 819	8 971	1 969 800	1 323 000	.....
1940	12 764	9 617	2 127 300	1 419 000	1 650 090
Extended at 7% 1941	13 657	10 011	2 276 130	1 477 000	.....
1942	14 613	10 421	2 435 460	1 537 000	.....
1943	15 635	10 848	2 605 800	1 600 000	.....
1944	16 729	11 292	2 788 120	1 666 000	.....
1945	17 890	11 754	2 981 610	1 734 000	2 191 090

Possible Storage Reservoirs

Table 15, prepared by the U. S. Bureau of Reclamation, gives information on reservoir sites on the upper tributaries. Of these sites, it may be stated that:

1.—The Ouray site will be expensive to develop, because of depth to bedrock and width between abutments.

2.—The Kremmling site is probably unavailable on account of interference with the Denver and Salt Lake Railroad.

3.—The Bedrock site commands a relatively small run-off.

TABLE 15.

Stream.	Reservoir site.	Raise in water surface, in feet.	Reservoir capacity, in acre-feet.	Average annual run-off at site, in acre-feet.	Foundation Conditions: Tests by	Foundation Conditions: Depth to bed-rock, in feet.
Green	Flaming Gorge	240	4 000 000	2 300 000	Drilling	73
"	Ouray	210	16 000 000	5 200 000	"	121
Yampa	Juniper	200	1 550 000	1 200 000	"	24
Colorado	Kremmling	230	2 200 000	1 200 000	"	104
"	Dewey	215	2 270 000	6 800 000	"	44
Dolores	Bedrock	210	800 000	400 000	None	....
San Juan	Bluff	206	1 350 000	2 300 000	"	....

The U. S. Bureau of Reclamation estimates the cost of developing four of these sites as shown in Table 16.

On the basis of cost as shown in Table 16 the average cost will be about $4.75 per acre-ft.

TABLE 16.

Reservoir.	Capacity, in acre-feet.	Cost.
Flaming Gorge	3 120 000	$16 000 000
Juniper	1 550 000	4 000 000
Dewey	2 270 000	11 200 000
Bluff	1 400 000	8 800 000
Totals	8 340 000	$40 000 000

The Flaming Gorge site is now under permit to the Utah Power and Light Company and will probably be developed in the near future.

The Juniper and Dewey sites can be developed for about 4 000 000 acre-ft. at a cost of about $4 per acre-ft.

In addition to these sites, there are possible reservoir sites at Glenn Canyon, Boulder Canyon, and Mohave, the capacities and areas of which are shown on Fig. 12.

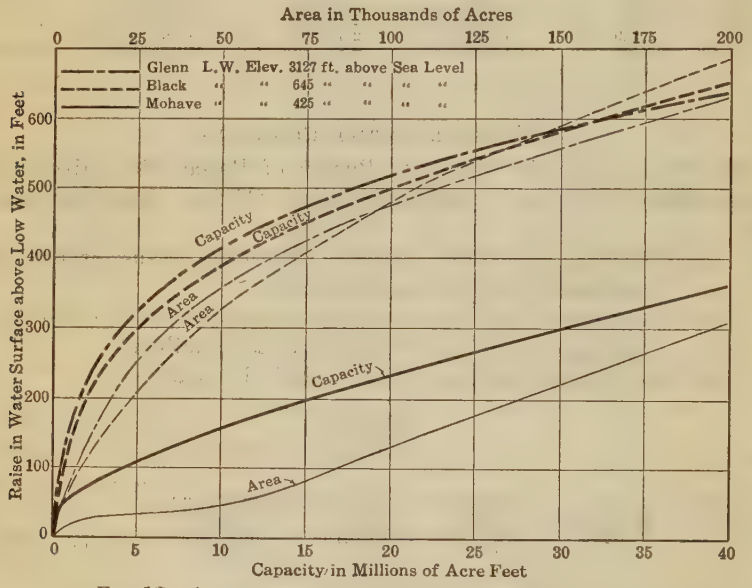


FIG. 12.—AREA AND CAPACITY CURVES FOR RESERVOIRS.

Glen Canyon.—This reservoir is near the head of the Canyon Section of the main river. The Southern California Edison Company explored the Glen Canyon site and found bed-rock at a maximum depth of 80 ft. The walls and bed of the canyon at the dam site are massive red sandstone, and the canyon walls rise almost vertical to heights of 1 000 to 1 500 ft. The engineers of the U. S. Bureau of Reclamation state that the sandstone is so poorly cemented that its safe bearing power will not be much more than 20 tons per sq. ft. Three

samples of core in compression tests failed at 2 220, 12 900, and 10 480 lb. per sq. in., respectively. The strength in compression of the weakest sample tested is, therefore, greater than that of mass concrete and, contrary to the views of the engineers of the U. S. Bureau of Reclamation, it seems to be safe to assume that the maximum stresses will be determined by the concrete and not by the bearing power of the stone in the foundations and abutments. The greatest objection to the site is its great distance (about 135 miles) from a railroad.

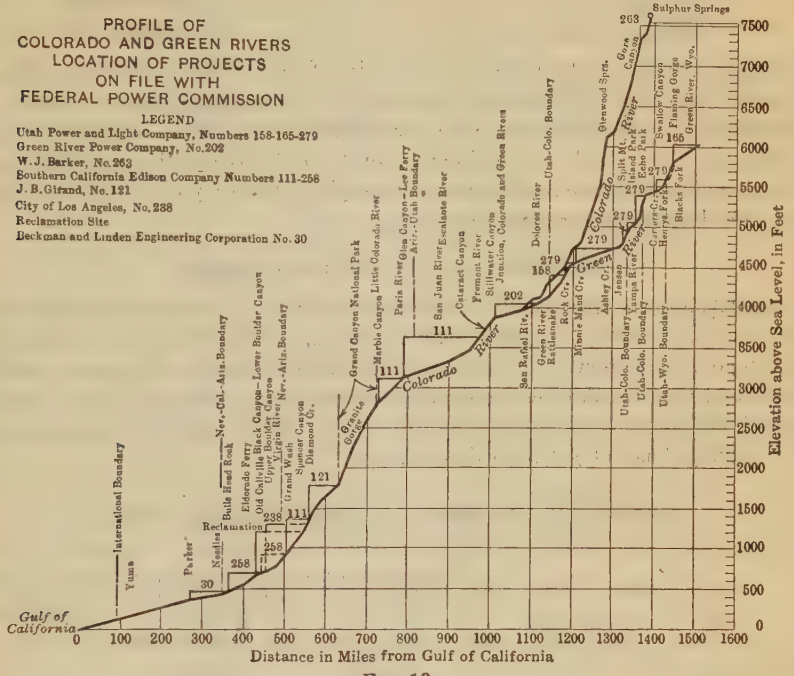


FIG. 13.

Boulder Canyon.—This reservoir site is near the foot of the Canyon Section. It has been extensively explored by the U. S. Bureau of Reclamation and is the site selected by it for a flood protection and irrigation reservoir for the Lower Basin. The project recommended contemplates a dam at a point in Black Canyon where the water surface is 645 ft. above sea level and the depth to bedrock is 124 ft. The dam will raise the water surface to 1 250 ft. above sea level and provide 34 000 000 acre-ft. of storage, the top 8 000 000 acre-ft. of which are to be reserved for flood protection and irrigation of the Lower Basin. The dam will develop 660 000 h. p. continuously with an average head of 550 ft. This project will be further discussed under the head of "Development Between Needles, Calif., and Diamond Creek".

Mohave Valley.—The Mohave Valley site is about 100 miles nearer the Lower Basin than Boulder Canyon and presents many other advantages as a site for flood protection and re-regulation for irrigation in the Lower Basin. It is easily accessible, develops great capacity with a relatively low dam, and leaves the entire Canyon Section of the main river free to be developed for

power. If satisfactory foundations are found, the only objection to the site is that its flowage damages may be high, as it will require moving the Town of Needles, a division point on the Santa Fé Railroad, and re-locating 15 to 20 miles of main-line railroad.

The U. S. Bureau of Reclamation has estimated the cost of providing 10 000 000 acre-ft. of storage at the previously mentioned sites, as follows:

Glen Canyon	$77 500 000
Boulder Canyon	27 000 000
Mohave Valley	26 000 000*

It also estimates that for a reservoir of 34 000 000 acre-ft. at Boulder Canyon, the 8 000 000 acre-ft. reserved for flood protection would cost $12 000 000, if the total cost were pro-rated on the basis of capacity used.

The data on which the estimates for Glen Canyon, Mohave Valley, and the upper reservoirs are based, are so incomplete that they constitute a doubtful basis for determining relative costs.

Possible Dam Sites on the Colorado

In 1923, a party from the U. S. Geological Survey went down the Colorado by boat from Lees Ferry, or Glen Canyon, Ariz., to Needles, Calif. This party made topographic surveys of sites that appeared favorable for dams, and it is understood that the data collected will be published. The list, shown in Table 17, includes some sites examined by this party. The depth to bed-rock at most of these sites has not yet been determined.

TABLE 17.—Possible Dam Sites, Colorado River.

Site.	Elevation of low water above sea level, in feet.	Probable limit of height of dam, in feet.	Distance, in miles, from Gulf of California.
Diamond	1 335	1 800	567
Bridge Canyon	1 207	2 000	557
Spencer Canyon	1 105	1 700	547
Devils Slide	1 034	1 330	537
Pierce Ferry	905	1 100	516
Boulder Canyon	705	1 250	459
Black Canyon	645	1 250	438
Bulls Head	496	650	372
Mohave	425	605	318
Laguna	151	.....	135

Development Between Needles, Calif., and Diamond Creek

Tables 18 and 19 have been prepared to show the extent to which the Boulder Canyon Project of the U. S. Bureau of Reclamation fails to conform to a full development of the Colorado River below Diamond Creek.

In Table 19, Plan 1 is the Boulder Canyon Project. It is contemplated that when further irrigation is developed in the Lower Basin, a re-regulating dam will have to be constructed at Bulls Head.

Plan 2 includes the Bulls Head Dam. The head between the Boulder Project and Diamond Creek (about 80 ft.) is too small to permit of economic

* Includes $13 000 000 for flowage damage.

TABLE 18.—Data on Boulder Canyon and Mohave Reservoirs.

Boulder Canyon Reservoir.

Maximum elevation of pool above sea level, in feet.	Minimum elevation of pool above sea level, in feet.	Height of dam above low water, in feet.	Mean head available for power, in feet.	Storage available for flood protection, in acre-feet.	Mean area of reservoir exposed to evaporation, in acres.	Evaporation loss per year at 5 ft., in acre-feet.	Present evaporation loss, in acre-feet.	Increase of evaporation loss, in acre-feet.	Acres that could be irrigated with water lost.	Horse-power available, (mean head x 1 210).
905	905	260	260	0	35 000	175 000	35 000	140 000	31 000	315 000
905	905	345	345	0	37 000	185 000	40 000	145 000	32 000	418 000
1 030	1 030	385	385	0	68 000	340 000	40 000	300 000	67 000	466 000
1 250	1 250	605	605	0	160 000	800 000	50 000	750 000	167 000	732 000
1 250	1 155	605	550	8 000 000	130 000	650 000	50 000	600 000	133 000	667 000
1 250	1 225	605	580	4 000 000	145 000	725 000	50 000	675 000	150 000	702 000
1 250	1 235	605	590	2 350 000	155 000	775 000	50 000	725 000	161 000	714 000

Mohave Reservoir.

425	.....	....	....	.....	.....	.....	.....	.....	.....	.....
525	425	100	....	4 340 000	68 000	340 000	310 000	30 000	7 000	0
525	470	100	....	4 000 000	71 000	355 000	310 000	45 000	10 000	0
545	425	120	....	6 125 000	68 750	343 750	310 000	33 750	7 500	0
545	490	120	80	4 000 000	72 800	364 000	310 000	54 000	12 000	97 000
582	425	157	....	10 000 000	88 000	440 000	350 000	90 000	20 000	0
605	425	180	....	12 800 000	97 000	485 000	350 000	135 000	30 000	0
605	530	180	125	8 000 000	98 000	490 000	350 000	140 000	31 000	151 000
605	545	180	155	4 000 000	113 000	565 000	350 000	215 000	48 000	187 000

Note.—Average evaporation loss, Topock to Yuma, for 1922-23, 1 770 000 acre-ft.; overflowed area, 306 000 acres. Overflow area: Mohave Valley, 62 500 acres; Cottonwood Valley, 9 500 acres; total, 72 000 acres. Present evaporation loss may be taken as: Mohave Valley, 310 000 acre-ft.; Cottonwood Valley, 40 000 acre-ft.; total, 350 000 acre-ft.

TABLE 19.—COMPARATIVE PLANS OF DEVELOPMENT OF COLORADO RIVER BELOW DIAMOND CREEK

(Assuming Up-River Storage Developed. Evaporation Loss at 5 Ft. per Year; Irrigation Duty, 4 to 5 Ft. per Year.)

Plan.	Dam.	Elevation of pool above sea level, in feet.	Head available for power, in feet.	Storage available for flood and irrigation, in acre-feet.	Evaporation loss, in acre-feet.	Horse-power available.	Acres lost on account of evaporation.	Cost.
			U. S. Bureau of Reclamation Plan.					
1	Boulder	1 250	550	8 000 000	600 000	637 000	133 000	$50 000 000
2	Bulls Head	645	0	1 650 000	25 000	0	.....	14 000 000
	Boulder	1 250	590	2 350 000	725 000	714 000	.....	50 000 000
	Totals	.....	590	4 000 000	750 000	714 000	169 000	64 000 000
3	Mohave	605	125	8 000 000	140 000	132 000	.....	27 000 000
	Boulder	1 030	385	.....	300 000	465 000	.....	27 000 000
	Devils Slide	1 330	300	.....	10 000	363 000	.....	22 000 000
	Totals	.....	810	8 000 000	450 000	960 000	100 000	76 000 000
4	Mohave	605	180	4 000 000	215 000	187 000	.....	27 000 000
	Boulder	1 030	385	.....	300 000	465 000	.....	27 000 000
	Devils Slide	1 330	300	.....	10 000	363 000	.....	22 000 000
	Totals	.....	865	4 000 000	525 000	1 015 000	117 000	76 000 000
5	Mohave	545	0	6 000 000	33 750	0	.....	24 000 000
	Black Canyon (Lower End)	905	345	.....	145 000	418 000	.....	26 000 000
	Pierce Ferry	1 112	207	.....	5 000	248 000	.....	17 000 000
	Spencer Canyon	1 330	218	.....	5 000	262 000	.....	18 000 000
	Totals	.....	770	6 000 000	183 750	928 000	38 000	$85 000 000

development, so that Plan 2 represents full development under the Reclamation plan.

Plans 3 and 4 contain similar structures, namely, a dam 180 ft. high at Mohave, a dam 387 ft. high at the Reclamation site in Boulder, and a dam 298 ft. high at Devils Slide. Plan 3 is comparable to Plan 1 and Plan 4 to Plan 2. The comparison shows that the Reclamation plan will cost $12 000 000 less, but will ultimately prevent the development of about 300 000 h. p. and 50 000 acres of land. The magnitude of this loss of power may be made clearer by pointing that the hydro-power generated in Southern California in 1923 was just about 300 000 h. p. If applied for under the Federal Water Power Act, the Reclamation project would be refused a license until it was modified to conform to a full plan of development. It should not be built by the Federal Government on the lines proposed.

Plan 5 is presented in order to illustrate the limit in curtailment of evaporation losses.

It should be stated in connection with the figures used in the comparative plans just mentioned, that power and evaporation losses of the Boulder Project are believed to be low and complete data probably will modify estimates of cost so as to wipe out more or less the difference of $12 000 000 and, as a flood protection and irrigation project, Mohave has so many advantages that it should be fully investigated before any other project is adopted. If it is found feasible, a site should be sought at the lower end of Black Canyon for a power dam sufficiently high to back water up to either Pierce Ferry or Devils Slide, and the relative feasibility of the two latter sites should determine the height to which the dam should be built.

Silt

The Colorado carries large quantities of silt. If a flood-control and irrigation dam at the lower end of the Canyon Section were the only one in prospect, large capacity would have to be provided for silt. With the Diamond Creek Dam ready to be built for power and other power dams sure to follow before Diamond Creek can become filled, there is no justification for a large increase in present expenditure to provide silt capacity. If the Mohave Reservoir is built to a capacity of 6 000 000 acre-ft., it will give liberal capacity for silt deposit.

Increase of Low-Water Flow Undesirable at Present

There is serious objection on the part of the upper States to a material increase of the low-water flow of the river until the Colorado Compact is ratified by Arizona, the only State which has not yet done so. This compact was negotiated for the purpose of determining by mutual agreement, rather than by litigation, the allocation of waters between the several States in the Basin. There is good basis for objection to the Boulder Project of the U. S. Bureau of Reclamation, because it would put to beneficial use for power practically all the water in the river and thereby acquire prior rights that might have to be purchased before any further irrigation up stream could be accomplished.

There would be little ground for objection if the project were reduced to provide only for present needs of flood protection and irrigation.

The Boulder Project would provide water in the Lower Basin at all seasons far in excess of present irrigation requirements. This water will pass into Mexico and there be used for irrigation. Once used, its withdrawal for use in the United States will be difficult, if not impossible. As heretofore shown, storage for flood protection will not obviate the need for maintaining levees and bank protection in Mexico, and a treaty to facilitate this work and to limit the quantity of water to be supplied Mexico should be negotiated before the quantity for irrigation in Mexico is increased.

Work Which Should Be Undertaken by the United States

There are now before the Federal Power Commission applications covering practically the entire Canyon Section of the river. (See Fig. 13.) All the development justified at present, including flood control and irrigation, will be undertaken by private capital under adequate Federal and State regulation if the Federal Water Power Act is left free to function.

If, notwithstanding this situation, Congress deems the special conditions on this stream such as to demand Federal construction of a flood protection reservoir, investigation of the Mohave site should be made at once and, if reasonable foundations can be found, the dam should be built there. In view of the objections to increasing the low-water flow at present, it would be best to construct this dam for purposes of flood control and regulation of flow for irrigation only and not for the development of power. It could then be operated so as to prevent the appropriation and use of water beyond present needs. If power is installed, there will be great pressure to operate for the power demand. There seems to be small justification for charging the entire cost of such a dam to the power consumers but, if Congress decides to do so, a dam 180 ft. high at Mohave will produce more than 130 000 h. p., which would justify an installation of about 240 000 h. p. Projects in Southern California have averaged about $130 per h. p. installed, but there are few developments left that can be built for less than $200 per h. p. installed. The estimated cost of the Mohave Dam is $27 000 000, which charged to 240 000 h. p., gives less than $115 per h. p. To this, add $35 per h. p. for cost of power equipment, and the total is $150 per h. p. installed. This price compares so favorably with prospective California developments that there is little doubt that the power can be made to carry the cost of the dam without raising present prices to consumers. If the Mohave Dam proves to be feasible, it can be built in 4 years as against the 8 years estimated for Boulder Canyon Dam.

Conclusion

The proposed high dam in Boulder Canyon should not be built because it will curtail ultimate power development by 300 000 h. p. and ultimate irrigation by about 50 000 acres.

All the development needed on the Colorado will be built by private capital under adequate Federal and State regulation if the river is given over to development under the Federal Water Power Act.

If the Federal Government decides to provide flood storage for the Lower Basin, it should be provided in Mohave Valley where it will conform to the maximum use of the river for all purposes. Such a dam should be designed and operated for the present in a manner that will not increase the low-water flow beyond present irrigation needs. Steps should be instituted as early as practicable to negotiate a suitable treaty with Mexico covering the maintenance of levees, canals, etc., in Mexico and an agreement as to the water that will be provided for Mexican lands.

DISCUSSION

J. C. Allison,* M. Am. Soc. C. E.—It has been well stated that an attempt to solve all the problems of the Colorado River at a single stroke on the meager data now available is likely to result in ill-advised expenditures if nothing else. Irrigation in the Lower Basin of the river is only twenty-two years old; in the Upper Basin it is not a great deal older and is less extensive. The idea of storage dams in the river canyons for the triple purposes of flood control, complete storage of water supply for irrigation and power, although not old in theory, is new in the respect that only recently has an attempt been made to investigate, by field survey, the actual elements involved in these three problems. On the strength of the relatively meager data at hand, the public expects a wholly untamed river to be harnessed completely with one dam, doing away with the necessity for the great systems of levees protecting the lower valleys; it expects every flat place shown on the map within the water-shed of the river (and a great deal of land outside the water-shed) to be irrigated immediately. Each State involved is naturally fighting for a right to the use of water on every acre possible, discounting the fact that perhaps only a small proportion of the acreage it is planned to irrigate will be found feasible for reclamation. International complications with Mexico are conjured up, with the result that great expenditures of money are planned solely for the purpose of holding the entire control of the flow of the stream in American territory. Suddenly, the power possibilities in the river are realized, and the fight for power rights is on. The only element in all the foregoing that requires immediate action, that is, the control of the floods sufficiently to save the lower valleys from annihilation, is almost smothered in the haste to solve all the problems of the river at one stroke.

Fortunately for all concerned, the Committee on Irrigation and Reclamation of the U. S. House of Representatives, which is investigating the Swing-Johnson Bill providing for the financing by the Government of works contemplated to solve all the Colorado River problems, has been wise enough to understand the importance of the problem and is studying all angles of the situation before reporting the Bill. As these involve purely an engineering analysis of the project, it is certainly the duty of every engineer to contribute all the information and study available to the data already assembled. In this connection, Colonel Kelly's paper is certainly most important.

Although all cannot agree with the author's conclusions in their entirety, most engineers can certainly agree that the immediate necessity for flood control is paramount, and that the Government should provide without delay the means for constructing a dam of sufficient proportions to minimize the flood dangers, leaving the multitude of remaining problems involved in complete river regulation for irrigation and power to be solved later. It is the purpose of this discussion to rectify a number of the conclusions drawn from insufficient data.

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Flood Control.—The lower irrigation projects of the Colorado River, including the Imperial Valley, the Yuma Valley, and the Palo Verde Valley, must continue their levee maintenance work until regulation of the river is complete. With the tremendous variation in the annual discharge from a low point of 9 110 000 acre-ft. in 1902, to as high as 25 400 000 acre-ft. in 1909, it must be evident that one dam can never perfectly perform the triple functions of providing maximum flood control, maximum conservation of water for irrigation, and maximum power development. The provision necessary for silting space alone should bar this unattainable feat from further consideration.

A dam in or below the Canyon Section having a storage capacity of 4 000 000 acre-ft. and discharge facilities of 75 000 sec-ft., should be sufficient to care for the immediate problem, this dam to be utilized later for re-regulation of the flow for irrigation when other dams for storage and power purposes are built above it. Such a dam meets all the requirements for immediate flood control noted by Colonel Kelly, with the possible exception of regulation to provide entirely for the Gila River floods. Unless the dam itself is located within a reasonable distance from Laguna Dam, above Yuma, Ariz., it is impractical to attempt any substantial regulation of the Colorado River supply to offset the flash discharges entering the river from the Gila at Yuma. Inasmuch as it is necessary to continue the maintenance of the levee system of the lower valleys, the floods of the Gila are automatically cared for until such time as its complete regulation is justifiable. The very fact that the duration of the Gila floods is so extremely limited, justifies the assumption that only local damage can be done to the lands in Yuma and Imperial Valleys in case of levee failure. The reasons for continuing the maintenance of the levee system in the lower valleys, as enumerated by the author, are conclusive.

Irrigation.—In the speaker's opinion the conclusion drawn by Colonel Kelly and by the engineers of the U. S. Reclamation Service, as well as by many of the State engineers interested in the project, that there are more lands to irrigate eventually than can be served by the water available, is erroneous. Only experience will determine this issue. If the speaker's judgment is correct, as the facts herewith indicate, a great many of the most serious problems involved in the regulation of the Colorado River disappear completely.

Table 20 illustrates the progress of irrigation and the net usage of water on the lands of Imperial Valley.

For a period of 5 years the average duty was 2.8 acre-ft. per annum, which can be safely applied to the other valleys below the Canyon Section. It has taken 22 years to reclaim and irrigate 731 893 acres below the Canyon Section, distributed as indicated in Table 21.

This relatively small total area represents the lands most easily irrigated during a period in which agriculture has had a great stimulant through the necessities of war.

TABLE 20.—Irrigation Data for Imperial Valley.

Year.	Acreage irrigated, Imperial Valley, United States, and Mexico.	Volume of water, in acre-feet, actually used on lands.	Actual water duty, in acre-feet.
1919	553 560	1 697 836	3.07
1920	607 000	1 792 484	2.95
1921	536 000	1 316 205	2.46
1922	565 000	1 474 999	2.61
1923	530 000	1 534 423	2.89

Familiarity with the lands in the Lower Basin, gained over a period of twenty-two years, indicates the following ultimate acreage below the Canyon Section (Table 22) as available for reclamation. By comparison with these lands, others are probably impracticable for any extensive irrigation, either because of altitude, lack of fertility, alkaline tendencies, or prohibitive costs of reclamation.

TABLE 21.—Areas Irrigated.

Imperial Valley, United States (measured by the actual lands covered by water stock)	448 893 acres
Imperial Valley, Mexico	190 000 "
Yuma Valley	54 000 "
Palo Verde Valley	35 000 "
Parker Project	4 000 "
Total	731 893 acres

Besides these 2 000 000 acres, there will be no demand for irrigation, except on high mesa lands where the return from agriculture will not justify the expense of an excessive pumping lift.

TABLE 22.—Ultimate Areas Possible of Irrigation Below Canyon Section.

Imperial Valley, Mexico (on both sides of the Colorado River)	740 000 acres
Imperial Valley, United States, within the present irrigation district	448 893 "
Imperial Valley, outside lands, East and West Mesas	323 000 "
Coachella Valley and Dos Palmos	100 000 "
Yuma Project	100 000 "
Palo Verde Project, including Cibola Valley	65 000 "
Above Palo Verde Valley, including Parker Project	90 000 "
Total	1 866 893 acres

Assuming a water duty of 2.8 acre-ft., with a 25% allowance for seepage, evaporation, and carrying losses to the points of delivery, there may be a total maximum demand in the Lower Basin, including Mexico, of 7 000 000 acre-ft. per annum. Considering that the greater portion of the additional lands that may come under irrigation in the Lower Basin are more difficult and expensive to reclaim, it is not likely that they will be brought under cultivation as quickly as the 731 000 acres now irrigated. However, assuming that the agricultural demand will necessitate the same rate, it will be prob-

ably forty years before the Lower Basin will demand the entire 7 000 000 acre-ft. of water per annum.

The Colorado River Compact, between six of the seven Basin States, provides 7 000 000 acre-ft. of water for the States in the Upper Basin. The maximum usage possible as determined in the paper is 4 500 000 acre-ft. plus 500 000 acre-ft. possible to withdraw into water-sheds other than the Colorado. It is doubtful whether such a quantity of water can ever be put to beneficial use in the Upper Basin States; but presuming a final usage of 5 000 000 acre-ft., the period over which this will be developed will be considerably more than the forty years necessary to complete the usage in the Lower Basin.

In spite of the withdrawal from the tributaries of the Colorado from the upper water-shed, of water to irrigate approximately 1 500 000 acres, as found by the U. S. Reclamation Service, the annual discharge of the Colorado River at Yuma for a period covering the past forty-four years indicates no traceable depletion of supply. The history of irrigation in Southern California and elsewhere during the past twenty-two years reflects a similar condition. As long as the water is not withdrawn from the water-shed, the quantity returned to the river after irrigation may be expected to more than offset any shortage caused by usage on any acreage contemplated. The speaker's experience on the projects in the Lower Colorado, in Imperial Valley, Mexico, and the Palo Verde Valley, especially the latter, where the return water was measured to some extent in a drain canal constructed along the river near the protective levee, indicates the tremendous return of irrigation water to the stream. With a completely regulated Colorado River, whatever the flow, or whether the complete allotment provided for in the Colorado River Compact can be utilized by the Upper Basin States, the Lower Basin projects will never suffer from water shortage as long as no considerable quantity of water is withdrawn from the water-shed. The following outstanding facts sum up the situation:

First.—Under complete regulation, there will be ample water for any future needs.

Second.—On account of the comparative slow advance of irrigation, and the apparent rapid advance of power demand, complete irrigation of all the lands available in the Colorado River water-shed will, without doubt, follow complete regulation of the river for power; power demand will stimulate a regulation of the river from which irrigation will later profit in ample time for its needs. As long as the power dams are designed and regulated by the Government to the ultimate end that all the water of the river can be properly conserved for irrigation usage at the times of irrigation demand, the one dam built immediately for partial flood control, and the additional dams built to meet the growing power demand, will ultimately completely solve the Colorado River problem and attain all the results sought in the Swing-Johnson Bill.

Third.—With an excess of water for irrigation, international disputes over the water are needless and a complete treaty understanding between the United States and Mexico is easily attainable.

All-American Canal.—To February 29, 1924, the Imperial Irrigation District had expended $511 357.60 for All-American Canal surveys, Boulder Canyon Dam surveys, lobbying and conference expenses, and in the Laguna Dam connection to the Imperial Valley Water System; by far the larger part of this expense was incurred through the desire on the part of politicians within the Imperial District to create an All-American Canal.

The argument for an All-American Canal dates from the execution of the contract of October 23, 1918, referred to by the author, between the Secretary of the Interior and the Imperial Irrigation District. Contrary to the interpretation placed on it by advocates of the All-American Canal, this contract does not provide for the construction of such a work; it provides principally for a connection of the Laguna Dam above Yuma with the Imperial Valley Canal System at the International Boundary Line to avoid the necessity of further temporary dams in the river at Hanlon Heading, the intake of the Imperial Canal System that is objectionable to the water users in the Yuma Project directly across the stream. It provides, secondarily, for a survey of the All-American Canal which, if found practicable by the Secretary of the Interior, was to be built by the Imperial Valley System, work commencing within a period of two years. The late Secretary Lane, the author of this contract, positively disapproved of building the All-American Canal on account of its impracticability, as found by the survey; hence, the contract in this respect is of no effect.

Colonel Kelly's misunderstanding of the contract of October 23, 1918, does not differ from that of a great part of the public interested in the project. The voters at the election on its ratification were voting for a connection with Laguna Dam to avoid a threatened water shortage and further disagreement with the Yuma water users; they were not voting for an All-American Canal.

The Engineering Profession is making a grave mistake in sanctioning, without protest, the building of a nationally known work on the location adopted for this so-called All-American Canal, especially without a more complete understanding of the necessities making such an enterprise even worthy of consideration. Although the engineers engaged in the design of this work have protested its limitation within the United States, their protest has not been so strong as to prevent the politicians and selfish interests from passing quickly over it and proclaiming the location of the canal as one freely chosen and sanctioned by the Engineering Profession. Thus, they have gained the confidence of the public and through this confidence have nearly succeeded in securing Government financing through what is known as the Swing-Johnson Bill. Just as surely as the promotion of the project has been tied to the engineers of the country, just that surely will its construction failures be fastened to the engineers unless they examine forthwith in detail all phases of the project and proclaim their findings in no uncertain terms. Then, if such a work is ever financed and built, it may be known as a political necessity and not as a sound engineering structure.

Colonel Kelly's position with regard to the building of the connection between Laguna Dam and the Imperial Valley Canal at the International

Boundary Line, is shared by the speaker. However, the so-called All-American Canal, designed to replace the main canal of Imperial Valley now running through Mexico, should not be built on any of the hypotheses set forth by its sponsors for the following reasons:

1.—It does not add one acre of land to irrigation that cannot otherwise be added at much less expense per acre and in greater acreage.

2.—It does not add one drop of water to the available supply, but, on the other hand, represents an extravagant waste through seepage and evaporation losses.

3.—It does not add one unit of power, but develops less electrical energy for the uses of the American lands than can be developed by the other means suggested herein.

4.—If at all a necessity, it must be a political necessity, and this political necessity is set up by its advocates as the principal reason for its construction. The fact of the present canal being in Mexico is the main reason advocated for the building of a duplicate canal on American territory. The principle, however, is unsound because, as will be shown, Mexico as a water customer is of strict financial value to the American users of water, and, as an ally and neighbor in the use of water, is of great political value in the project. As a matter of fact the new project does not in any sense relieve the owners of the present Mexican Canal from maintaining their flood and irrigation works.

5.—Above all, the construction and the maintenance difficulties and the prohibitive expense of the All-American enterprise warrants a most careful examination before it shall receive the endorsement of the Society in any form. The reasons for this statement will now be discussed.

It is proposed under the All-American project (shown on Fig. 19) to bring water from the Laguna Dam (after raising and modifying the dam to care for the additional diversion) through an enlarged canal located along the route of the present Government canal as far as what is known as Siphon Drop; thence continuing along the foot-hills between the mesa and the river bottom-lands as far as Pilot Knob, where it is to divert westerly along the International Boundary Line and plunge directly into the forbidding territory between Pilot Knob and the Imperial Valley. The plan calls for the abandonment of the gravity system now supplying Imperial Valley through Hanlon Heading at Pilot Knob by way of Mexico. This present supply flows entirely below the surface of the ground at Hanlon Heading and affords a safe delivery of water to Imperial Valley through all seasons of the year. Any desert rain storms along the route of the proposed canal from the Laguna Dam to Pilot Knob can, without difficulty, break it and divert its entire supply back into the Colorado River. The storm regulators provided by the engineers are inadequate to cope with the inevitable cloudbursts draining from the mesa lands along the foot of which the canal is to run. Although the new canal from Laguna Dam should be constructed as far as Pilot Knob, as projected, still the diversion through Hanlon Heading, directly from the river and into the Valley by its present route, should be maintained as a safeguard.

The Imperial Irrigation District, with a population of 60 000 dependent entirely on the flow from the river, cannot be subjected to the possibilities of delays in water service.

It is proposed, from Pilot Knob westerly, to construct the All-American Canal in a cut ranging from 40 to 160 ft. in depth, through 20 miles of high mesa land and 12 miles of the most extensive drifting sand-hills in America (Fig. 14) along the approximate route of the spectacular California State Highway between Yuma and Imperial Valley (Fig. 15), which is built of portable plank board sections, for raising and lowering the grade and regulating the route with the drifting of the sand. Along this same approximate route is constructed the power line of the Southern Sierras Power Company; its experience alone in maintaining 12 miles of line should be enough to weaken the determination of the most enthusiastic supporters of an All-American Canal. One day 30-ft. power poles are covered to the wires with sand and the next day undermined by a 24-hour sand storm.

To build a canal through this section is in itself a most infeasible exploit, not only because of the difficulties of excavation and original construction, but also on account of the hazards and maintenance expense afterward.

Fig. 16 shows a cross-section of this work at one of the deeper cuts. It is proposed to make this excavation with suction dredgers at a cost of 20 to 30 cents per cu. yd. Experience shows conclusively that heavy sands and gravels cannot be pumped at this figure even with low lifts, let alone through the gigantic lift of 150 ft. and more which will obtain when the spoil banks are built up along the edges of the cut. No consideration is given the fact that it is an impossible feat to excavate with hydraulic machines, maintaining such a slope as is estimated; probably 50% of excess materials must be moved if this means of excavation is utilized. Should even 20% of the excavated materials be waste, the cost of the cut will be increased by between $2 000 000 and $3 000 000. Should the engineer's estimate be exceeded by 5 cents per cu. yd., the cost of the work will be increased $3 000 000. Not only is the work itself a dangerous experiment, perhaps an entirely impossible task, but the estimates themselves of such an experiment are undependable, as the whole project is at the mercy of the elements.

The velocities that will obtain in the canal as planned range from 2½ ft. to 3½ ft. per sec.; these velocities are as high as the engineer dared risk in the loosely knit sand and gravel through the mesa, yet a velocity of 4 to 5 ft. per sec. will not keep in suspension the surface sands which blow in from the surrounding country. In other words, if a velocity is maintained sufficient to keep the drifting sands in suspension and avoid the clogging of the canal, then, at certain seasons of the year, when the silt content from the river water is light, this same velocity will erode the banks in a manner that will ruin the entire section of the canal.

A canal all on American territory, designed to serve the western slope of Imperial Valley, would pass along the boundary line north of the City of Calexico, crossing New River on an extensive viaduct, costing $915 000 with its wasteway, and enter the West Side Main Canal, which is on the high ground

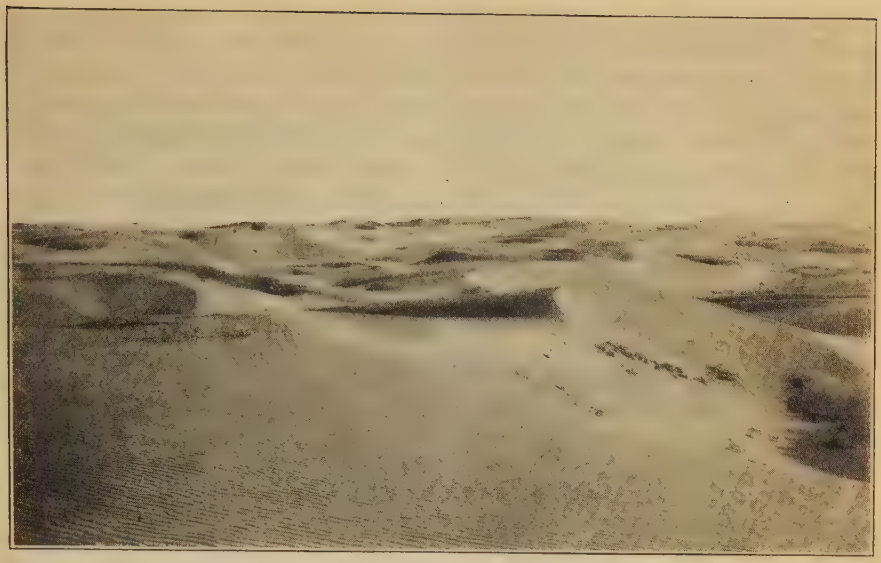
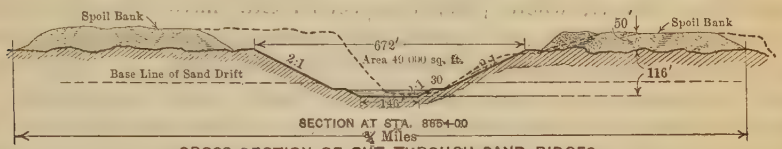


FIG. 14.—DETAIL OF SAND-HILL REGION TRAVERSED BY ALL-AMERICAN CANAL, SHOWING "BARCHAN" FORMATION.

FIG. 15.—LOCATION OF ALL-AMERICAN CANAL THROUGH THE SAND-HILLS, SHOWING ALSO THE CALIFORNIA STATE PLANKED HIGHWAY ALONG ITS APPROXIMATE ROUTE.

of the western slope of the Valley. The fills required crossing the western section of the Valley to attain the altitudes necessary on the western slope, would be as high as 15 ft. above the natural ground. This section of the country is already menaced by ground-water due to seepage from the local canals and if the All-American Canal were carried across this same territory, at such an extraordinary height, this section would be ruined from ground-water. A considerable area of land west of the present West Side Main would be left in its present condition permanently, on account of the low elevation of the All-American Canal.



CROSS SECTION OF CUT THROUGH SAND RIDGES
SHOWING PROBABLE CHANGE IN SECTION AT END OF ONE YEAR

0 100 200 300
SCALE IN FEET

FIG. 16.

Assuming the All-American Canal to be built, the next consideration concerns its future maintenance. The very fact that the stream is in a cut 50 ft. below the floor of the mesa itself, with crowning sand-hills of heights ranging to 100 ft. above the floor, makes it absolutely certain that the cut itself will receive and hold the entire volume of sand drifted to it by each storm. Where the sand-drift passes across the country without interruption, ofttimes a pass through the sand-hills, such as the Government Pass in which part of the canal is located, can remain fairly open; but where the passage is deliberately interrupted by a cut having a flowing stream at its bottom, then all the sands must necessarily enter and remain in this excavation.

Here, again, Imperial Valley must depend on its present canal system through Mexico if interruptions in the delivery of water are to be avoided—interruptions that unquestionably will be fatal on account of the huge quantity of drift involved.

The borings taken along the route of the proposed canal indicate porous strata in the canal prism. Especially with the Boulder Canyon Dam built and with some relief from the silt thrown into the canals as at present, this prism cannot possibly seal itself. The water losses from seepage, as estimated from the losses occurring in the present East High Line Canal through similar sections of material, will amount to 27%, unless the canal is lined with concrete. This loss is prohibitive not only because of the interference in regulating the supply to the land about 150 miles distant, but more particularly because the very water lost to the All-American Canal will mean the ruination, from saturation, of all the low-lying lands along the section of the country through which it passes.

As an alternative, lining the canal with concrete or tunneling the sand-hills is impractical, principally because the section must be constructed for the maximum irrigation requirement immediately, as it will be difficult to

enlarge. As it may be twenty years before the entire capacity will be demanded for irrigation, the additional investment at present is prohibitive.

However, the main objection to the location of such a canal is the uncontrollable elements injected into the problem in deliberately cutting and maintaining a waterway through the very heart of a desert mesa region, capped with drifting sand dunes. By means of studies herein discussed, these mammoth sand dunes are easily recognized as similar in size and construction and in the phenomena governing their movement, to the most extensive types the world over. As the welfare of thousands of souls in the oasis of the great Sahara Desert, where prosperous settlements have been overwhelmed and blotted out of existence, is intimately connected with the rate of movement and the mode of accumulation of wind-borne sands, so is the fate of one of the greatest irrigation regions of the world coupled inseparably with a similar movement of great sand-hills on the route of the All-American Canal. The lessons taught by engineers and geologists of England, France, and the United States, in their years of study and observation of the sand dune regions of the world, must be applied in this case to avoid disaster in attempting to build the All-American Canal.

In addition to the difficulties of construction, there is the question of determining from experiments elsewhere whether or not the sand-hills through a 12-mile section of the All-American Canal can be prevented from drifting into the canal; secondly, it remains to determine the rate of movement in the sands of the character found in this case; and, thirdly, having projected a determined quantity of sand into the canal, it is important as to whether or not conditions can be set up in the canal itself enabling the water to transport the sand without resorting to mechanical means.

Although in localities of lesser sand-drifts, fences, trees, and the oiling of the ground surface are useful in stopping their encroachment, under conditions of such overwhelming drift, as in the case at hand, no such means would be successful. The only effect of oiling or planting vegetation is to create other sand dunes. Trees, brush fences, and such obstacles placed in the path of such extreme drift are most readily submerged, emerging, however, as the dunes move on. In the case of the All-American Canal, as the sand rolls over such obstacles, it next encounters the running stream of water which is the only effectual means of stopping it. It follows that the most general method of checking the encroachment of blow sands is to promote the growth of sand dunes; and a sand dune is best promoted by a running stream. Sand-hills on the western shores of the Holy Land, and in a district extending almost from the borders of Egypt, including the neighborhood of Alexandria, are stopped only by the Nile. Sand dunes are invaders far more dangerous than the waves of the sea and only a stream that will carry off the particles will effectively stop them.

In determining the capacity of the canal to carry off the sands injected into it, it is necessary to examine the mechanical elements of the materials involved and to determine the quantity to be removed. Table 23 indicates the remarkable similarity in the grading of sand from dunes scattered over the United States. The similarity of sands taken along the route of the All-

TABLE 23.—Mechanical Analyses of Sands.

Samples from the Route of the All-American Canal.

Sample No. 1* Taken near Station 800 from Bottom of Sand-Drift.			Sample No. 3* Taken near Station 800 from Top of Sand-Drift.			Sample No. 7* Taken near Station 980 from Sand-Flat.			Average percentage of three samples.
Diameter, in millimeters.	Percentage.	Proportion, in ounces.	Diameter, in millimeters.	Percentage.	Proportion, in ounces.	Diameter, in millimeters.	Percentage.	Proportion, in ounces.	
2 — 1	.....	.....	1 — 2	.....	.....	1 — 2	3.10	2.50	1.03
1 — ½	21.25	17.00	½ — 1	5.62	4.50	½ — 1	55.60	44.50	27.49
½ — ¼	40.00	32.00	¼ — ½	46.25	37.00	¼ — ½	19.40	15.50	35.21
¼ — ⅛	38.75	31.00	⅛ — ¼	48.13	38.50	⅛ — ¼	21.90	17.50	36.26

Samples from Similar Sand-Drifts in Different Parts of the United States.†

Diameter, in millimeters.	Rear slope of a dune, Alliance, Nebr.	Rear slope of a dune, Tampico, Ill.	Rear slope of a dune, New Boston, Ill.	Rear slope of a dune, Alliance, Nebr.	Rear slope of a dune, Alliance, Nebr.	Rear slope of a dune, Hooppole, Ill.			Sand-drift, Griggs Company, North Dakota.	Sand-drift.	Average percentage.
16 — 8	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
8 — 4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4 — 2	.....	.....	0.20	.....	.....	.....	.....	.....	.....	.....	Tr.
2 — 1	9.00	3.90	1.80	0.20	0.10	.....	0.20	.....	.....	0.30	1.55
1 — ½	80.	43.80	22.00	37.10	33.70	12.40	9.70	6.20	19.30	14.00	27.84
½ — ¼	6.60	28.60	46.20	50.90	49.40	34.60	44.40	39.20	45.20	42.90	38.80
¼ — ⅛	5.00	22.00	26.20	18.90	15.00	33.00	40.70	29.40	30.70	37.90	25.88
⅛ — 1/16	0.10	1.70	2.80	1.70	0.10	17.20	4.00	23.40	2.40	2.50	5.59
1/16 — 1/32	.....	.....	0.20	.....	.....	2.00	0.30	1.20	0.40	1.80	0.59
1/32 — 1/64	.....	.....	.....	.....	.....	.....	.....	.....	Tr.	0.30	Tr.
1/64 — 1/128	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
1/128 — 1/256	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

Comparison of Averages.

Diameter, in millimeters.....	1 — 2	½ — 1	¼ — ½	⅛ — ¼
All-American Canal.....	1.03	27.49	35.21	36.26
Similar sand-drifts.....	1.55	27.84	38.80	25.88

* See Fig. 17 for position of samples. Five pounds of each sample used. Analyses by J. C. Allison, M. Am. Soc. C. E.

† Analysis taken from "Mechanical Composition of Wind Deposits", by John August Udden, 1898.

American Canal to samples taken elsewhere in the United States, and the mechanical analysis of the sands of the Lybian Deserts and the regions of the Sahara, justify the assumption that the phenomena governing the movement of sands in the older and more carefully studied regions, will be repeated in the case at hand. In Fig. 17 is shown the positions from which the speaker's samples on the All-American Canal were taken.

WIND ACTION ON SAND DUNES OR "BARCHANS"
Figures indicate location from which samples of Table 23 were taken

FIG. 17.

The scouring action of fluids varies theoretically as the square of the velocity and so does the diameter of the particle transported. If the velocity is doubled, the diameter of the particles transported may be increased four times. The transporting power of wind varies approximately with its erosive force, that is, as the square of the velocity, as in fluids, whereas its lifting capacity changes according to the sixth power. The range of velocity of dune-making winds certainly exceeds twice the normal, and hence it might be expected that the bulk of sand, in some places at least, would consist of grains many times as large as in others, were it not for scarcity of the larger material.

Size alone may make a dune a permanent hill even if it is composed of loose sand throughout. Given a constant climate, a large desert dune might easily outlast the highest mountains, for the denuding agency continually renews the surfaces. On the other hand, there is necessarily a limitation of the process whereby dunes grow, which prevents their attaining heights equal to those of mountains formed by erosion. Winds have greater power at considerable elevation than at the surface so that more sand is removed from the summits than is replaced by the wind and thus the lowering of the summits is not offset by the deepening of the trough; action on the summits of dunes is hastened, and in the trough hindered, by gravity, and should a trough be created by excavation through the sand dunes, the phenomena mentioned would continue until the point was reached by the filling of the trough where any deepening would be compensated by the lowering of the summits.

Obstacles, such as a wall, affect the distribution of sand by wind in two ways, directly as obstacles and indirectly by affecting the motions of the air. Wind evokes eddies on each side of an obstacle. These eddies on the weather side pick up the finer grains which those on the lee side capture, throwing away only the finest of them. Sand, therefore, deposits against both sides of the wall. A high canal spoil bank, even though wetted, oiled, and grown with shrubs, acts merely as a wall, the sand being deposited both on the outside and on the canal side. If the sand is fine and the wind strong, the principal deposit is on the lee side or in the canal; if the wall or embankment is high, nothing at first will get over it, except dust wafted in the air. In time,

however, the mechanical obstacle offered by the bank results in building up a sloping platform on the weather side and when the platform reaches sufficiently near the top of the embankment, the winnowing action described commences. The deposit on the lee side being formed by sedimentation takes its angle of repose, usually quite steep. If, however, by favoring conditions, the sloping platform on the windward side should in time surmount the bank or wall, so that the large grains could roll over, a proper dune or "barchan" profile is produced (Figs. 14 and 17), and the movement of the dune or barchan into the canal corresponds with the movement of the dune of similar character without the area of the canal prism.

In the observations made by Mr. H. J. L. Beadnell, formerly of the Geological Survey of Egypt, on the dunes of the Lybian Desert, the following results were obtained with an air meter under different conditions of wind in the dune belts (from personal observation, corresponding results are obtained in the sand-dune regions of the All-American Canal):

1.—On the dunes, the sand commences to move when the winds attain the velocity of a light breeze, or 13 miles per hour.

2.—On the open plains, the wind becomes visibly charged with sand as soon as the velocity of a moderate breeze (23 miles per hour) is obtained.

3.—Sand-storm conditions obtained when the wind exceeded a moderate breeze, the maximum velocity actually recorded being 34 miles per hour.

The following experiments correspond in general to those made in an attempt to trace the annual movement of a sand dune through the section of the proposed All-American Canal:

1.—According to Dr. Franz Czerny, dunes on the Baltic Coast in the Province of Courland moved at the rate of 18 ft. per year.

2.—Bromontier measured the movement of the dunes of Gascogne at the rate of 25 m. per year.

3.—Cornish found the average rate of advance of a crest of a dune near Ismailia to be about ¾ in. per hour.

4.—Mr. C. R. Enock estimated the movement of barchans in South America to be as much as 2 ft. per hour in a brisk breeze.

5.—On the Lybian Desert, the movement of barchans is as much as 25 m. per year.

6.—In the sand dunes along the route of the All-American Canal, movement of barchans as great as 85 ft. per year has been recorded.

Fig. 18 shows a profile of that section of the proposed All-American Canal through the sand-dune region. It is reasonable to expect from the experiments mentioned that the sand dunes through which the canal is cut will shift an average of 80 ft. per year. The cross-section of the canal (Fig. 16), indicates as nearly as possible, what effect this shifting will have on the canal prism. A calculation of the quantity of sand shifted per year into the canal, as reflected in the cross-section, gives 3 136 000 cu. yd.

The mechanical analysis of the sands shifted into the canal corresponds to that of materials dragged along the bottom of Colorado River and like

streams. These materials form themselves on the bottom of the streams into dunes and anti-dunes and move as such in sand waves down the stream. It is due to the dragging rather than to the suspension faculties of the stream that these types of sands are transported. The first movement begins at a surface velocity of 1.3 ft. per sec.

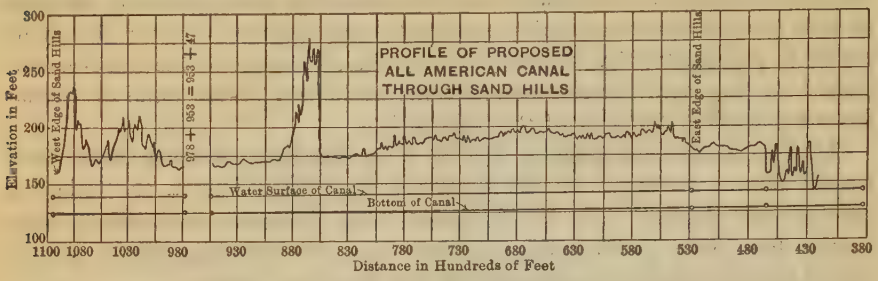


FIG. 18.

From the tabulated data on movement of sand waves,* the following cases are recorded where velocities and sand weights correspond to those of the All-American Canal: In the Mississippi River at Helena, Ark., crests of sand waves moved 17.17 ft. per day with a stream velocity of 3.58 ft. per sec. In the Loire, for corresponding velocities the movement was from 16.4 ft. to 36 ft. per day. Measured by M. Sainjon, in the Loire, under a surface velocity of 1.33 m. per sec., the crests of sand-bars moved from 0.000056 m. per sec. to 0.000087 m. per sec.

From the foregoing experiments it is safe to assume that materials of like nature injected into the All-American Canal will be dragged along the bottom of the canal in sand waves under the prevailing velocities of 3 to 4 ft. per sec., a distance of approximately 6 000 ft. per year. Of the materials presumed to have been thrown into the canal by the wind, only one-tenth will be thus scoured from the critical section through the sand-hills, leaving 2 800 000 cu. yd. to be removed with dredges each year. As a matter of fact, even the small proportion that passes through the critical section in sand waves will only be distributed through lower reaches of the canal, as there are no sluiceways available near the sand-hill section. Any neglect in the yearly dredging of the canal or any accumulation of sand deposits in one section will result in a complete blocking of the water supply, unless the Mexican Canal is maintained as a stand-by. The effect will be to demoralize the living conditions of the 50 000 to 60 000 people in the Imperial Valley, who are dependent entirely on the Colorado River for their water supply.

Practical Substitute for the All-American Canal.—The outlying lands in the Imperial Valley Basin, including the East and West Mesa and the Coachella Valley, can secure a water supply from the Colorado River. However, this must be attained without overburdening the already too heavily taxed users of water in the Imperial Irrigation District. This can be done in a practical manner and at much less expense than the All-American Canal

* "The Suspension of Solids in Flowing Water", by Elon Huntington Hooker, *Transactions*, Am. Soc. C. E., Vol. XXXVI (1896), p. 239.

route, for the new mesa lands and the Coachella Valley. These new Government lands can then be acquired and farmed successfully by the returned soldier, or by any other entrymen who can finance and build their own canal systems at the proper time, in the same manner as the settlers of the other districts within the present Imperial Irrigation District and in Coachella Valley without the necessity of Government appropriation.

Fig. 19 shows the proposed route of the All-American Canal and lateral canal system for the new lands, as covered in the report referred to. Delineated also on the same map is the system proposed hereunder, to irrigate the same lands, except in greater acreage. Briefly described, this proposal is as follows.

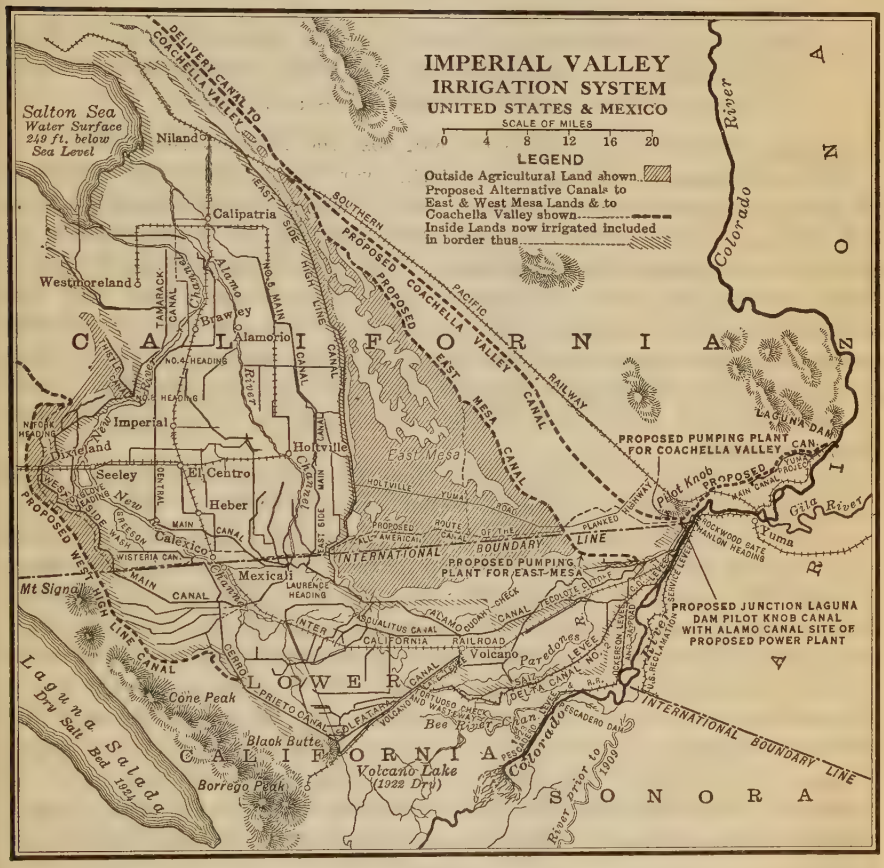


FIG. 19.

Laguna-Pilot Knob Canal for Imperial Irrigation District.—Construct the canal and works from Laguna Dam to Pilot Knob as provided in the All-American Canal report. At Pilot Knob, empty the entire flow of the canal into the Alamo Canal, utilizing the present main channel through Mexico.

Construct a power plant at the point of delivery into the Alamo Canal to utilize the Mexican, as well as the American, water to generate 43 700 h.p. This power is calculated from the following factors: Delivering water from Laguna Dam to Pilot Knob at an elevation of 143.87 ft. and dropping 13 000 cu. ft. per sec. 37 ft. into the Alamo Canal. This water can shortly be sold to American and Mexican lands during part of the year and used to sluice the Imperial Valley Canals during the remainder of the year. The power developed will be delivered as follows:

8 500 h.p. to the Yuma project, as per contract of October 23, 1918;

7 500 h.p. to lift 1 300 sec-ft. of water from the new canal to the floor of the mesa at Pilot Knob to irrigate 100 000 acres of land in Coachella Valley and at Dos Palmas;

21 500 h.p. to lift 2 600 sec-ft. of water from the Alamo Canal to the floor of the mesa at the west end of the sand-hills to irrigate 200 000 acres of East Side Mesa lands in United States and Mexico.

This proposed canal and power plant will cost the District $5 600 000.

The advantages in this proposed work, over the All-American proposal, are that the Hanlon Heading Diversion, and the Alamo Canal through Mexico, will serve as a stand-by for the water system to Imperial Valley in case of mishap to the Laguna Dam connection. The power development is immediately greater than that developed by the All-American Canal, and the market for it is at hand, with little transmission loss and expense. As the irrigated acreage increases on either side of the International Boundary Line, so will the power development increase, while with the All-American Canal no future increase can be had and the Mexican water power is lost to the District entirely.

Canal for Coachella Valley.—Lift from the Laguna-Pilot Knob Canal 1 300 sec-ft. of water for Coachella Valley and Dos Palmas. This water is to be lifted from an elevation of 143 ft. to the floor of the mesa at an elevation of 183 ft. and carried in a canal on top of the mesa in a normal cut along the northeasterly side of the sand-hills (Fig. 19), at a distance to avoid sand-drift or encroachment, to the main line of the Southern Pacific Railroad, at an elevation of 128 ft. Here, it crosses and follows a grade contour to Coachella Valley 37 ft. higher than that reached by the All-American sand-hill route. The route on this location is 13 miles shorter than by way of the sand-hill route and its higher elevation places it to greater advantage as far as storm drainage is concerned. The higher contour irrigates 23 000 acres more in Coachella Valley than the sand-hill route, making in all 100 000 acres of agricultural land in place of 77 000 acres as contemplated. The costs would be as follows:

Pumping plant at Pilot Knob	$ 500 000
52 miles of canal, Pilot Knob to Iris	1 200 000
Iris to end, including laterals	6 474 000
Total	$8 174 000

For 100 000 acres this would be at $81.74 per acre, as compared to $102.00 per acre, estimated by the All-American Canal report. The saving in maintenance on the pumping route will more than pay for the power and operation of the pumping plant.

Canal for East Side Mesa Lands.—Lift from the Alamo Canal in Mexico, at Elevation 93, 2 600 sec-ft. to the top of the mesa west of the sand-hills, at Elevation 160, to irrigate all the East Side Mesa lands, as contemplated in the All-American sand-hill route (160 000 acres in the United States and 30 000 acres in Mexico), together with 10 000 acres not reached by the All-American route. To lift this flow from the Alamo Canal, a 20-ft. head pumping plant would be required, the water would be carried in a Mexican canal for a distance of 5 miles to the foot of the mesa, where it is lifted in two or more lifts 47 ft. to the top of the mesa; the canal on the mesa follows along the westerly edge of the sand-hills (Fig. 19) to Iris, on the approximate route of the pumping canal lateral contemplated in the All-American program. The cost would be as follows:

Pumping plant, 20-ft. lift	$ 500 000
Pumping plants, 47-ft. lift	850 000
Main Canal to Iris, plus 100 000 acres of lateral system	4 466 800
Plus additional 100 000 acres of lateral system at $16	1 600 000
Total	$7 416 800

Under the All-American sand-hill route the cost per acre is $85.20, with a cost for maintenance which will operate the pumping plants under the Mexican route.

Canal for West Side Mesa Lands.—Construct a canal from the Cerro Prieto Canal in Mexico, at Tule Check, following a grade contour along the western slope of the valley above the line contemplated by the All-American Canal System, irrigating agricultural land as follows:

Lands covered by All-American Canal System	20 000 acres
New lands above All-American Canal System	10 000 "
Pump lands covered by All-American Canal System	23 000 "
Total	53 000 acres

The cost of this project would be as follows:

Enlarging Solfatara and Cerro Prieto Canals in Mexico, 28 miles	$105 000
Head-works at Tule Check	80 000
New canal to International Boundary Line (19 miles)	200 000
Building "B" Line Canal on new location plus enlargement for additional acreage	641 000
Total	$1 026 000

For 53 000 acres	$19.35 per acre
Plus distributing system	16.00
Total cost for gravity lands	$35.35 per acre
Plus pumping system	50.50
Total cost for 23 000 acres of pumping lands	$85.85 per acre

Under the All-American Canal System, the corresponding costs are $106.00 per acre for the gravity lands on the West Side and $156.50 per acre for the pump lands.

The advantage of the Mexican route for the West Side lands, is the saving in cost per acre, the saving in maintenance, and in more direct and simple routing, over the prohibitory expense and impossible location of the All-American Canal System as proposed.

Mexico.—As has been pointed out, supporters of the All-American Canal must necessarily create a plausible reason for building it. They cannot well use the merits of the canal itself which are by comparison negative. Therefore, the only remaining argument for the canal is Mexico, the "bogey-man". There being no grounds in truth for an argument they and their supporters have recklessly resorted to imaginary accusations which have no bearing on the situation whatever, and, in their attacks on Mexico, have exposed the interests of American farmers in the Imperial Valley to the possibility of a retaliation that might do irreparable damage to the Nation.

Among other accusations, the supporters for the All-American Canal contend that Mexican lands have not paid their proportion of the expenses necessary in maintaining the canal and flood-control works of the Imperial Irrigation District. Mexican lands are not owners of these works as are the purchasers who happen to be the Imperial Irrigation District, but are merely customers, having no representation in the control of water affairs and having no participation in the possible earnings from the Canal System. Tables 24 and 25 indicate the amounts which Mexico has overpaid in the past five years in the water and flood-control affairs of the Imperial Irrigation District.

In Table 24 the entire financial distribution of the funds of the Imperial Irrigation District has been studied and, from the figures presented by the Auditor of the District, extracts have been taken. The four years of Irrigation District affairs ending December 31, 1922, have been chosen, as those years will properly emphasize the financial relationship between the Mexican and the American sides of the line; the first 1½ years of the financial history of the District (previous to 1919) reflect approximately the same relationship, but the actual distribution of figures is not dependable as to segregation between Mexican and American expenditures.

In the four years covered by Table 24, to avoid the risk of dispute as to expense and disbursement, all the expenses of the Imperial Irrigation District, on both sides of the line, have been taken into account, and Mexico has been charged with its proportion of these expenses. In reality, Mexico is simply a water customer of the Imperial Irrigation District and of its Mexican Branch, known as "Compania de Terrenos y Aguas de

TABLE 24.—Imperial Irrigation District—Statistics from Financial Reports, 1919-1922.

	1919	1920	1921	1922	Total.
Acreage irrigated, United States	417 000.00	417 000.00	414 000.00	415 000.00	1 663 000.00
Acreage irrigated, Mexico	136 560.00	190 000.00	122 000.00	150 000.00	598 560.00
Total acreage irrigated	553 560.00	607 000.00	536 000.00	565 000.00	2 261 560.00
Mexico irrigated 26.47% of entire acreage for 4 years. In the same manner, Mexico irrigated 26.55% of entire acreage in 1922.					
Total interest paid by Irrigation District on all District bonds and warrants, together with all discounts, taxes, and other similar charges paid, except discount on fourth issue of bonds sold for purchase of Mutual Water Companies and drainage in United States part of District	$451 025.73	$534 552.97	$458 316.09	$408 555.00	$1 852 449.79
Mexico's proportion of total interest charge is 26.47% of $1 852 449.79, or $490 343.46. In the same manner, Mexico's proportion of interest charge in 1922 is 26.55% of $408 555, or $108 471.35.					
Water sales, United States	$699 782.20	$814 867.54	$699 690.60	$788 952.59	$3 003 292.93
Water sales, Mexico	340 710.66	499 000.00	360 702.33	469 740.64	1 670 153.63
Total water sales, United States and Mexico	$1 040 492.86	$1 313 867.54	$1 060 392.93	$1 258 693.23	$4 673 446.56
Mexico paid 35.74% of all water revenues for 4 years. In the same manner, Mexico paid 37.32% of all water revenues in 1922.					
Total operating costs of District on both sides of Boundary Line for both canal and levee systems, includes superintendence, maintaining canals, levees, bridges, structures, buildings, fixtures, grounds, power and telephone lines, insurance, maintenance of all machinery, including depreciation; cost of distributing water; salaries and expenses of Irrigation District directors, officers, clerks at all points, law expenses, advertising expense, etc.	$932 221.68	$1 001 776.53	$623 683.50	$898 795.63	$3 456 477.34
Profit in water rentals over operating costs	108 271.18	312 091.01	436 709.43	359 897.60	1 216 969.22
Other profits, exclusive of tax assessments, certificates of redemption and interest on same, or any other profit paid by taxation on assessment by United States side of line	54 040.61	99 245.00	59 701.36	93 356.64	306 344.01
	$162 311.79	$411 336.41	$496 410.79	$453 254.24	$1 523 313.23

TABLE 24.—(*Continued.*)

Mexico's proportion of total profits based on the amount of money she paid in to secure said profits, is 35.74% of $1 523 313.23	$544 432.15
Mexico's proportion of all interest as per above	490 343.46
Mexico's over-payment of all operating costs and interest for four years	$ 54 088.69
In the same manner, Mexico's proportion of total profits for 1922 is 37.32% of $458 254.24	$169 154.49
Mexico's proportion of all interest charges for 1922 is	$108 471.35
Mexico's over-payment of all operating costs and interest for 1922 is	$ 60 683.14

la Baja California, S. A."; the original costs and maintenance of all the canals and structures north of the International Boundary Line, with the exception of the Hanlon Heading and Laguna Dam structures, should not be charged against Mexico. If this were done, the overcharge which Mexico has paid on operating expenses, principal, and interest on bonds, etc., would be more than double that shown by Table 24.

The investigation summarized in Table 24 indicates the following facts:

1.—During the four years considered, Mexican lands paid in cash not only their acreage proportion of the operating costs of all work and expense incurred by the District, but also $54 088.69 in excess of their acreage proportion of all interest on bonds, warrants, taxes, and every other expense incidental to the capital investment of the District on both sides of the line.

2.—Again, for the year 1922, Mexico has paid in cash all its share of the operating costs on both sides of the line, together with its share of all the interest items for capital investment of the District on both sides of the line, and has paid in excess, $60 683.14. There should be added to the latter item, the sum of $68 020.06, paid in cash by the Colorado River Land Company, S. A., in 1922, on the levee work of the District in Mexico, making an over-payment by Mexico of more than $128 703.20.

To the total of both these items should be added a differential arrived at, as follows:

The Irrigation District virtually delivered water to the individual user on the American side of the Line, inasmuch as it allowed an average of 10% for seepage and evaporation losses in the Canal System, besides maintaining most of the main canals. In Mexico, the water charge is made directly at the main canal of the District, with no allowance for seepage and evaporation and for canal maintenance. Another item to the advantage of the American side, is the fact that none of the Water Companies paid the District in advance for the water; in fact, most of the Water Companies have been in arrears several months at a time, whereas the Mexican users pay in advance, before water is put into the Mexican Canals. Therefore, to equalize the differential in water charges alone, there should be added to the water payment of Mexico in four years the sum of $167 015.36, making Mexico's total over-payment for the four years, $221 104.05. In the same manner, the over-payment made by Mexico for 1922 should be $128 703.20 plus $46 974.06, or $175 677.26.

It will be noted that the expense per acre for water used in Mexico greatly exceeds that for the United States; for instance, the total cash

cost to Mexico per acre irrigated for 1922 was $3.13, while in the United States it was $1.09. This is not because Mexico used more water per acre than the United States, but because of the 10% differential allowance for seepage and evaporation in the United States and because the water charge by the District in Mexico was greater than that in the United States until the time the Water Companies were absorbed. The highest rate paid in Mexico for water is $2.08 per sec-ft.; the lowest rate in Mexico is 75 cents per 1 000 cu. m., or $1.83 per sec-ft., both without any allowance for losses. In the United States, the highest rate paid, until the date of absorption of the Water Companies, was $1.70 per sec-ft. It is true that since then this rate has been raised to $2.00 per sec-ft., but the water is delivered to the individual irrigator, eliminating all losses of whatever nature in canal transit; besides this, it is the practice now, and has been for some years, for the District to furnish water on the American side for canal sluicing, washing out alkali lands, and generally for helping out the individual farmer. No such courtesy is now or ever has been extended to the Mexican water user. The cash allowance, besides water, to the Water Companies by the District for 1920, 1921, and 1922, was $202 674.23.

As stated in the foregoing, none of the canal systems in Mexico, other than the main canal of the District, is maintained by the District. The amount of the assessment against a Mexican acre is the same or more than the average water assessment of the Mutual Water Companies in the District for the same purpose in the United States. As matters now stand, since the District has absorbed the Mutual Water Companies, to equalize the maintenance and water charges the District should also assume the obligation of maintaining all canals in Mexico. It should pay at the rate of $9.09 per acre for each acre irrigated in Mexico, together with the cost of the dredgers and other operating equipment of the Mexican Canal Companies and should assume the delivery of water to the individual Mexican user in the same manner that it has done with the Mutual Water Companies on the American side. To the water user on the American side, the District, in 1923, paid in cash the sum of $4 724 612 to cover the items of canal systems, equipment, etc.

The advantages to the American water user in the present system of selling water by the District to Mexico have never been properly presented by the District. Were the All-American Canal to be built, for instance, and should the District discontinue the usage of its Mexico main canals and the sale of water to Mexico en route, the American water user would not only have the increased bonded debt on his land, created in the financing of such canal, but he would bear the burden of the decreased revenues of the District. The District, in that case, would be identical with a large department store, having all the departments idle except the top floor; in other words, a store supplied with goods and equipment to do business on all ten floors, but only selling and creating a profit on the top floor. The expense of carrying water from the Heading on the river for fifty miles through Mexico, or through an All-American Canal on the American side, without creating any business until

it reaches the upper portion of the Line, or the irrigable area on the East Side Mesa in the United States, is a similar example.

The tax rate on the American side for the four years under discussion is, as follows: $1.67 in 1919; $2.35 in 1920; $1.85 in 1921; and $1.90 in 1922; or an average of $1.95. Should there be wiped out the revenues collected in advance from Mexico for the four years, in the total sum of $1 670 153.63, then the tax rate per acre to the American land and property owner would be nearly doubled.

In the studies which have been made of the entire question, the relationship of capital investment between Mexico and the United States has also been investigated. In the building of the canal and levee systems there has been constructed and utilized for the benefit of American lands, a total of 67 miles of canal and 55 miles of levee on the properties of the Colorado River Land Company, S. A., alone, totaling an acreage right of way (exclusive of lands lost by seepage damage, etc.) of 30 000 acres. This taken with the right of way occupied on the remainder of the Mexican lands other than those of the Colorado River Land Company, brings the total usage of property up to 50 000 acres, which, at $50 per acre, has a total capital value of $2 500 000, equivalent to more than Mexico's proportion of the bonded debt of the Imperial Irrigation District.

Table 25 is a summary taken from the figures furnished by the Imperial Irrigation District showing the over-payment by Mexico of its share of expenses for 1923. Summarizing, the actual amount of money expended to deliver water to the International Boundary Line, at the various points of diversion into the United States, for agricultural use, is as follows:

All diversion works at Hanlon Heading, etc.	$122 095.78
All costs of delivering Mexican water, and carrying American water through Mexico	185 986.50
All river protective work of every nature	197 966.68
Total cost to the Irrigation District to deliver water at the various canal points along the International Boundary Line for usage in the United States, and to deliver Mexican water	$506 048.96
Total water sales in Mexico	$526 971.56
Difference between money received from Mexico and total monies expended by the District	$20 922.60
Plus cash paid by the Colorado River Land Company for river protective work, not included in the above	$39 855.65
Total net profit to the Imperial Irrigation District, after delivering all Mexican water, and delivering American water to the various canals at the International Boundary Line	$60 778.25

In other words, during 1923 the Imperial Irrigation District furnished water to its American water users at the International Boundary Line absolutely free of cost and with an added cash profit of $60 778.25. Were the All-American Canal built and functioning in 1923, the same operating costs as those given, in the total of $506 048.96, would have to be met by the American water user in addition to all the other taxes, water costs, and expenses he paid in 1923.

Besides, the abandonment by the District of its 137 miles of canals through Mexico, and its 71 miles of protective levees and railroads, together with the rock quarries and properties pertaining thereto, amounting in all to a total capital value of approximately $6 000 000, would be lost.

Furthermore, in considering Tables 24 and 25, it should be remembered that at the time the Mutual Water Companies on the American side were absorbed, they owed the District a total sum of $573 109.23. The Mexican water users owed the District nothing; in fact, they paid in advance on a great deal of their water. Of the sum owing the Imperial Irrigation District by the American Mutual Water Companies, $202 674.23 was written off by the District, and this bonus, granted the American users, is charged in the financial statements proportionately against Mexico also.

The supporters of the All-American Canal contend that the restrictions placed on them and on the construction and operation works by Mexico are prohibitive. The restrictions in Mexico are far less severe than those in the United States wherever the public interest is involved. At no time in the history of irrigation of the Imperial Valley have the Mexican engineers been anything but helpful in the construction and maintenance of the System.

In addition to the innumerable other objections to having the canal system in Mexico, the supporters of the All-American Canal urge their desire to control the quantity of water Mexico shall receive from the Colorado River, in order that Mexico shall be the one to be deprived of water when a water shortage comes on them. The position alone of Mexico's irrigable lands in the Delta of the Colorado River assures them of an ample supply of water even if the entire regulated flow is utilized in the projects above them, as long as the waters are not withdrawn from the water-shed of the stream. The return waters from the irrigation of 6 000 000 acres in the projects above will furnish the comparatively small area of Mexican land with an ample supply of water forever. As there is a surplus of water for irrigation requirements, there should be no strife between the two countries in the division of the water, and a treaty arrangement whereby American water can be passed through Mexico under complete control of American owners should be made easy.

Conclusion.—Summarizing the whole situation, it must be evident after a careful study of the Colorado River problem from an unselfish and unbiased point of view, that outside of political considerations and ambitions there is no logical reason for the present agitation.

TABLE 25.—THE IMPERIAL IRRIGATION DISTRICT—STATISTICS FROM FINANCIAL REPORT FOR 1923.*

	1923
Acreage irrigated, United States	350 000
Acreage irrigated, Mexico	180 000
Total acreage irrigated, United States and Mexico	530 000
Mexico irrigated 33.96% of entire acreage in 1923.	
Total interest paid by Irrigation District on all District bonds and warrants, together with discounts, taxes, and other similar charges paid, except discount and interest in 1923, on fourth issue of bonds sold for purchase of Mutual Water Companies and drainage in United States part of District	$437 500.00
Mexico's proportion of total interest charge is 33.96% of $437 500, or	148 575.00
Water sales, United States	914 832.15
Water sales, Mexico	526 971.56
Total water sales, United States and Mexico	$1 441 803.71
Mexico paid 36.55% of all water revenues in 1923.	
Total operating cost of the Irrigation District for 1923, including diversion work from the Colorado River to the International Boundary Line, all expense and maintenance costs in Mexico for both American and Mexican water, all river protective work, all expense of the Board of Directors and General Administration on both sides of the Line, all storage and All-American Canal expense, all telephone and power-line expense, etc., the above includes every expense of every nature, except only construction cost of the drainage system in the United States, plus construction costs and costs of delivering water to the individual American ranchers, after the water crosses back into the United States in the various American lateral canals	$799 177.89
Profit in water rentals over operating costs for 1923	642 625.82
Other profits entering the Mexican "Companía" funds, but excluding profits entering "General Funds", such as interest on bank deposits, property sales, etc	61 848.77
Total profits from operation for 1923	$704 474.59
Mexico's proportion of total profits based on the amount of money Mexico paid to secure said profits, is 36.55% of $704 474.59, or	257 485.46
Mexico's proportion of all interest and discount charges on bonds as per this statement	148 575.00
Mexico's over-payment on operating costs and bonded debt costs for 1923 is	$108 910.46
Add to the above, cash payment to the Imperial Irrigation District by the Colorado River Land Company for its proportion of river protective work, in 1923, as per agreement (this sum not shown in the District's 1923 Financial Statement because it was paid in the early part of 1924)	39 855.65
Mexico's total over-payment of its proportion of all costs for 1923	$148 766.11

* All charges for maintenance and operation of every nature (including All-American Canal expense to date), have been taken into account and charged proportionately to Mexico, except only (*a*) interest and discount on fourth issue of District's bonds sold to purchase Mutual Water Companies and to build drainage works on the American side; (*b*) monies expended in distributing water to individual American water users after it passes into the United States; (*c*) monies expended in building drainage works in United States; and (*d*) capital purchase of buildings, fixtures, and equipment for use in distributing water to individual water users in the United States.

If fully conserved there would be more water in the Colorado River watershed than is needed by the practical agricultural land on both sides of the International Boundary Line. It does not require an All-American Canal to serve the remaining agricultural lands in Southern California. A flood-control dam, built forthwith at some point on the river by the Government will solve the immediate flood problem, leaving the Federal Power Commission free to act in granting power permits to complete the development of the river after the Colorado River Compact between the States has been ratified.

FREDERICK H. FOWLER,* M. AM. SOC. C. E.—Present engineering information on the Colorado River problem has been ably summarized by the author. Several of his conclusions are diametrically opposed to a plan that has received wide publicity and has secured a firm hold on the popular imagination, the so-called "Boulder Canyon Project". Although the speaker does not regard this project as the best solution of the Colorado River problem, he admires most heartily its bold conception as an individual development.

In its final form, the Boulder Canyon Project proposes a dam rising 605 ft. above low water, capable of storing 34 000 000 acre-ft. behind it, the dam itself to cost at least $50 000 000. The power rights are to be leased to political sub-divisions or to others on a basis that will amortize the cost in fifty years. The power plants have been estimated to cost $36 000 000, and the transmission lines leading to the market, $46 000 000; a total of $132 000 000, or more, without the All-American Canal that has been appended to the project in pending legislation. According to its proponents, this project would:

(*a*) Reduce the maximum flood flow to 40 000 sec-ft.;
(*b*) Furnish a regulated irrigation supply to all irrigable lands below it; and
(*c*) Produce 660 000 continuous horse-power with a surplus even more than this at certain times.

The author holds that the development is not justifiable, because:

1.—No marked economic gain will result from reducing the flood flow below 75 000 sec-ft., and regulating the discharge of the Colorado River proper to 40 000 sec-ft. will result at times in increasing its flow during the winter months when floods may be expected from the Gila, and, in this manner, greatly increase flood danger between Yuma, Ariz., and the Gulf.

2.—The irrigation supply for lands below the Grand Canyon will ultimately be insufficient even with the tremendous storage proposed; and, for all lands in the basin to be irrigated until 1940, 2 300 000 acre-ft. of storage will suffice.

3.—The large block of power made available immediately on the completion of the dam cannot readily be absorbed by the prospective market, and there will be, therefore, a serious accumulation of interest charges on the unproductive part of the investment.

4.—The construction of the project will result in the permanent loss of 300 000 h.p. and of an irrigation supply sufficient for 50 000 acres.

The radical difference in these two sets of conclusions is due primarily to the differences in the plan of development as proposed in the Boulder Canyon

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Project and by Colonel Kelly, and, secondarily, to the author's reduced estimate of available water supply.

As regards the difference in plan: The Boulder Canyon Project qualifies as desirable under the Reclamation Act, and is actually covered by a Special Act, but does not qualify under the Federal Power Act. Colonel Kelly's plan, however, will qualify in every particular under the Power Act by giving the maximum beneficial development of the entire river.

The Federal Water Power Act was passed by Congress on July 10, 1920, after nearly ten years of debate. If it is permitted to function properly, it should prove in the long run a most valuable piece of legislation. It is far more than the mere "Water Power Act" its name would imply, for it was drafted to accomplish the well-ordered development of rivers, not only for power, but for the joint maximum benefit from irrigation, power, flood control, and navigation. One of its many requirements is that any project adopted shall be such as will be best adapted to a comprehensive plan of improvement and utilization of a stream for the purpose of navigation, of water-power development, and of other beneficial uses (including, of course, irrigation and flood control). It provides further that, if necessary in order to secure these results, the Federal Power Commission shall have authority to require the modification of any project.

Certain other provisions of the Federal Water Power Act have direct bearing on the question of State or municipal *versus* public utility ownership—the issue that is being fought out on the Colorado. These provisions are:

1.—That licenses under the Act are not given in perpetuity, but have a limited term.

2.—That the Act gives preference to municipalities, provided their plans are equally adapted to the best ultimate development of a stream.

3.—That adequate rentals are to be charged for developments made by individuals, public utilities, or by States and municipalities (when operated for a profit), and part of the Government rentals are to be returned to the State or States within which the power development is located.

All these items bear directly on the "public *versus* corporate ownership" phase of the controversy and show that the Special Act confers only one important "benefit" not included in the Federal Water Power Act, namely, an initial appropriation of $70 000 000 from the United States Treasury. Moreover, there is nothing in the Act which prevents the Federal Power Commission from including, as part of the terms of any license, the so-called "Colorado River Compact" or any part thereof.

It is to be hoped that, after long years of discussion, having passed a comprehensive Act, Congress will allow it to function properly and will not hamper its operation by passing special legislation affecting the Colorado, as well as other important streams such as the Columbia, St. Lawrence, and Tennessee Rivers, all of which fall in the same general class.

The inadvisability of special Federal legislation for a situation better covered by general legislation is only the first objection to the proposed Act; the second is that the financial plan is not sound.

It was originally proposed that preliminary expenses in connection with the Boulder Canyon Dam be "charged on a basis of 85% against the interests that might consume power or for power purposes; 10% against flood control; and 5% against irrigation." This distribution was made arbitrarily, but its general theory was correct. Later, it was recommended as not only possible, but desirable, that the expense of the dam itself be carried entirely by the power interests—a proposal that naturally received more hearty popular support than the former, but was by no means as sound economically. In other parts of California land is being irrigated, drained, and protected from floods on a basis requiring payment in proportion to the benefits received. This principle is sound, and it follows logically that the lands on the Colorado should also pay in proportion to the benefits received from the storage of water. The same careful methods could be used in assessing such benefits as are now being used by the California Reclamation Board on the Sacramento and San Joaquin Drainage District. The various users of the stream would then be shouldering their own burdens.

Under the present Boulder Canyon plan, the power users are expected to pay the entire cost, and although it is reasonably certain that they can, there are grave doubts as to whether they should be required to do so.

"The cities and irrigation districts of Southern California" were referred to in the original Boulder Canyon report* as the most important market for Boulder Canyon power. Of the other possible outlets for power listed therein, the most important and readily accessible appear to be the mining regions of Arizona with the principal towns adjacent, and possible railway electrification on the Santa Fé, Salt Lake, and Southern Pacific Railways. Railway electrification is, however, entirely in the future, and its extent is dependent chiefly on the policy of the respective railroad companies. It is generally understood that the management of the Santa Fé System is in favor of electrification at the earliest possible moment, whereas the management of the Southern Pacific is distinctly antagonistic to all such plans. The immediate market for any large amount of power, therefore, is reduced to general uses in Southern California and mining uses in Arizona.

Assuming that "the cities and irrigation districts of Southern California" do buy most of the power, what will be the proper charge?

Why, for instance, should the irrigator of the Riverside region or the Lower San Joaquin Valley pay a price for Colorado River power that will vouchsafe both flood protection and free irrigation supply to his fellow irrigator who happens to be operating in the Palo Verde, Yuma, or Imperial Valley Districts?

Part of the power will go to Arizona. Why should the mining industry of Arizona pay a higher price in order that the limited areas along the river may receive free flood protection and irrigation supply?

It may be argued that the development of these lands will be of benefit to the nation, but it is obvious that, under the existing plan, owners of these

* Senate Document No. 142, p. 19.

lands will reap the benefit—and at the expense of other parts of the community.

Among the strongest engineering counts against the Boulder Canyon Project are:

1.—That it will curtail the ultimate development of the stream.

2.—That, until its large block of power is absorbed, it will continue to pile up carrying charges.

3.—That, after its power is absorbed, the part of the investment chargeable against regulative storage must be duplicated by expenditures for headwater storage unless the upper stream is to remain unregulated. Such duplication would lead to great economic loss.

The best plan of development on any stream is to concentrate the regulative storage as near the head-waters as it can be made effective, except in so far as this program has to be altered on account of local conditions. The burden of proof is squarely on the shoulders of any one who desires to depart from this basic rule, and no adequate justification has been made for the departure proposed on the Colorado.

Two very distinct flood control and storage problems are found on the Colorado:

1.—Reduction of the high and long sustained floods resulting from melting snows in the basin above the San Juan, and the storage of these waters for power and irrigation.

2.—Control of the high, but short or torrential, floods originating in the basins of the San Juan, Little Colorado, Virgin River, and minor tributaries of the plateau region. (The Gila is excluded from this list as it is below all proposed storage.)

It has been generally assumed in certain former studies that large storage for flood control and irrigation combined must be established at the lowest possible site on the Colorado in order: (*a*) To give protection from torrential floods of the San Juan, Little Colorado, and Virgin Rivers; and (*b*) to give closer regulation for irrigation than could be secured from head-waters storage.

Colonel Kelly's demonstration that little economic gain can be secured by reducing flood flows below 75 000 sec-ft. permits a considerable reduction in the flood-control storage needed on the lower river; and installation of headwater storage for control of the main Colorado flood would permit still further reduction in the capacity of the down-stream reservoir.

In view of the interesting flood records secured during September, 1923, it appears that the flood danger from torrential tributaries (other than the Gila) has been greatly exaggerated. As nearly as can be judged from combined stream-flow and rainfall records, it appears that on September 17 and 18, 1923, heavy precipitation occurred on the head-waters of the San Juan and over practically the entire basin of Little Colorado River. Two flood peaks resulted, as shown in Fig. 20. These floods were measured by river observations at Goodridge, Utah, on the San Juan River, and at Lees Ferry, Bright Angel, Topock, and Yuma, Ariz., on the Colorado. No flood was

recorded at the stations above the San Juan. The flood from the San Juan reached a peak at Goodridge about 6 A. M., September 19, and this peak traveling down stream reached Lees Ferry 23 hours later, or at about 5 A. M., September 20. Continuing down stream, it caused a secondary peak at the Bright Angel Gauging Station in a flood already discharging from the Little Colorado.

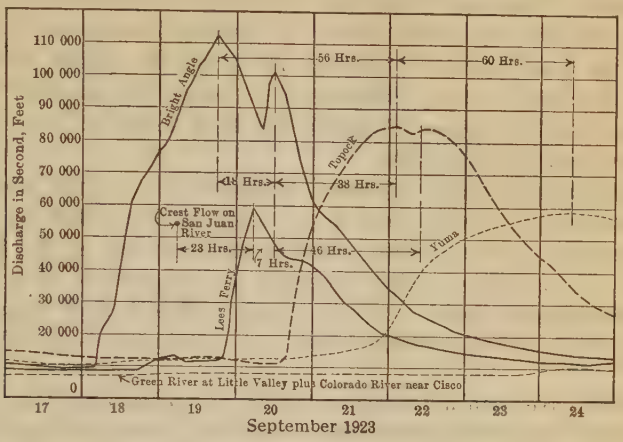

FIG. 20.

No observations are available showing exactly when the flood from the Little Colorado reached the main stream, but its peak had traveled down the main river and had reached the Bright Angel Station on September 19, at 6 P. M., 18 hours before the peak at the same station due to the San Juan flood. The discharge of the first peak at Bright Angel was about 111 500, and that of the second, 101 000 sec-ft.

The hydrograph at Topock shows that the first peak arrived at 2 A. M., September 22, or 56 hours later than at Bright Angel, and amounted to only 85 000 sec-ft.—a reduction of 25 500 sec-ft. The second peak was reached at 10 A. M., September 22, or 46 hours after the second peak at Bright Angel, and was 84 000 sec-ft., or 17 000 sec-ft. less than at the upper station. The records of both these stations were for fractional parts of a day.

The record for Yuma is based on daily averages, but the two peaks had probably flattened into one general rise, reaching a maximum discharge at about 10 A. M., September 24, or 60 hours later than the first peak at Topock. The Yuma record must be corrected for diversions at Laguna Dam and for inflow of the Gila,* but it happened that on the peak day these two discharges

* The mean daily discharges of the Gila and the diversions at Laguna Dam, September 20 to 25, 1923, were, as follows:

		Discharge of Gila, second-feet.	Diversions at Laguna Dam, second-feet.
September	20	200	2 072
"	21	8 000	2 020
"	22	3 900	1 091
"	23	3 000	954
"	24	2 000	2 237
"	25	1 500	2 155

practically balanced. It is probable, therefore, that the Yuma peak did not exceed 60 000 sec-ft. in any event, 25 000 sec-ft. less than at Topock. It appears, therefore, that with no regulation whatever a flood flow of 111 500 sec-ft. at Bright Angel was cut 26 500 sec-ft. by the time it reached Topock, and 51 500 sec-ft. by the time it reached Yuma.

Considering the great distance between the junctions of the San Juan, Little Colorado, Virgin, and Gila, and also the great difference in topographic conditions and elevations in the basins of these tributaries, and the different seasons of the year during which they are normally subject to floods, it is difficult to conceive of any combination of circumstances that would produce a flood from the upper tributaries reaching Yuma at the same time as an extensive flood from the Gila.

It would seem justifiable, therefore, to reduce the storage capacity of the flood reservoir on the lower river to a minimum far below that generally advocated to date, and to secure corresponding control in the head-waters. Even with this reduction, when working in conjunction with head-water storage, the reservoir would certainly supply sufficient local irrigation regulation and water for all projects likely to be completed in the immediate future.

Arguments have been advanced on behalf of the Boulder Canyon location which are really (especially when taken together) strong arguments against it and in favor of head-water regulation. For instance, in the first report on the Project, it was stated that "any large reservoir on the Colorado must depend for its financial feasibility upon the availability of an adequate market for not less than 500 000 h.p. electrical energy within economic transmission distance." The logical conclusion is that a large storage at Boulder Canyon will absorb the ready market for power, and that as all down-stream needs will be safeguarded, adequate regulation of the middle and upper river will be delayed until the dim and distant future.

In the same report the relative advantages of a great reservoir and power plants at Boulder Canyon as against Glen Canyon were carefully compared, and the Glen Canyon Project was condemned on account of its remoteness from the market, as well as for its less effective control of the flood and irrigation supply. No adequate program was advanced, however, for up-stream regulative storage operated in conjunction with down-stream power development and minor re-regulation.

This plan is now advanced by the author, who has shown clearly that it will furnish flood control and an irrigation supply for the same area as the Boulder Canyon Project, and that it will furnish a regulated power discharge through three times as much river descent. It will also reduce the hazard and cost of construction of the five or more dams necessary to develop the fall in the Canyon Section of the river—this last advantage is no small item in its favor. Under his plan, head-water storage and power development on the middle and lower stream can proceed gradually as conditions demand, the first step being only that necessary to give, in conjunction with minor storage on the lower river, the necessary flood-control and irrigation supply.

The functions of the lowest reservoir may be merely supplementary to the major storage in the head-waters, and its size, therefore, should be kept at a minimum, otherwise the investment will be so great that the reservoir must be operated independently of the head-water regulation, in which case, instead of adding to, it will detract from, the flexibility of the entire system.

Eliminating the Glen Canyon Project from consideration for the present by no means exhausts the possibilities of head-water storage. Flaming Gorge, Dewey, and Juniper Reservoir sites offer excellent opportunities. Their combined storage capacity would be 6 940 000 acre-ft., the average annual run-off past the dam sites, 10 300 000 acre-ft., and their total cost, $31 200 000. The prospects are good for the early development of Flaming Gorge without cost to the Government. The Dewey site could be developed by the Government, or by other agencies, at an estimated cost of $11 200 000, and would serve for power regulation throughout the Canyon and for control of floods from the Grand River. The development of the Juniper site for combined irrigation, power, and flood control, might be made at a later date.

The possibility of storage at the Mohave site is a comparatively new but important factor in the Colorado River problem. The Boulder Canyon site was formerly believed to be the lowest point on the river where large storage could be obtained, and this was properly advanced as a particularly desirable feature both because it gave additional flood protection and made possible closer regulation and avoidance of waste in the irrigation supply. It is now found that the original information was not complete, that there is ample storage at the Mohave site, providing the foundation conditions are satisfactory. It must be admitted that any advantages of position claimed for the Boulder site will apply with much greater force for the Mohave site. The Mohave site is 120 miles nearer the principal irrigation diversion, and its construction, according to the plans proposed by Colonel Kelly, will eliminate the necessity of constructing a re-regulating reservoir at Bulls Head. It will flood certain railway improvements, but, even allowing compensation for these, it is attractive financially. It will also flood certain irrigable lands, but these too are covered by the estimated damages, and, as the water supply is now shown to be inadequate for the full development of all available acreage, the elimination of this relatively small portion will not result in an ultimate reduction of the total area served.

The importance of securing more detailed information on the Colorado, before adopting a final program of development, was recognized and emphasized in 1924 by a plank in the Republican platform. It is to be hoped that at an early date funds will be provided, the investigations completed, and the development of the stream started on a comprehensive plan under the Federal Water Power Act.

E. C. La Rue,* M. Am. Soc. C. E.—Because of his investigations along the Colorado River during the past fourteen years, and his publication of considerable engineering data on the subject, the speaker is especially interested in all papers which deal with a plan of development for this great

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river system. The author has analyzed the engineering data resulting from recent investigations on the river without fear or favor. His conclusions regarding a plan of development appear sound. In his discussion, the speaker will deal only with the physical facts as they relate to the various plans of development, and will not express opinions as to the agency which should develop the water-power resources of the river.

During the past thirty-four years, engineers of the U. S. Department of the Interior have been investigating the water resources of the Colorado River Basin. Including its work in the Grand Canyon in the fall of 1923, the U. S. Geological Survey has mapped 1 800 miles of the river and its tributaries. All known dam sites on the main river, between Wyoming and Mexico, have been surveyed. Investigations made both by the Geological Survey and the U. S. Bureau of Reclamation, show that, with complete development, nearly 6 000 000 acres of land can be reclaimed in the American part of the Basin, and about 6 000 000 h.p. can be developed.

It is also well known that property in the Lower Basin, valued at more than $100 000 000, is menaced by the annual floods of the Colorado River. Practically every one agrees that the flood menace should be removed by providing storage somewhere on the lower river. A report that is being prepared by the U. S. Geological Survey, will show the relative value of all dam sites on the lower river, their relation to each other, and to the full utilization of the water resources.

Some fundamental facts should be recognized by all. The resources of the Colorado River are not unlimited. When its millions of horse-power have been developed, the end of water-power production in the southwestern part of the United States will have been reached. All the water resources of the Basin will be required for the irrigable lands within, or adjacent to, the Basin in the United States. If these resources are to be developed without waste, it is necessary that a comprehensive plan be made and agreed on before construction work is started.

As explained by the author, the Boulder Canyon Project does not conform as a unit to the full development of the river. This is not surprising, for this project was selected and approved before field investigations were made to determine the proper plan of development. However, the Boulder Dam has not been built, and it is not too late to prepare a comprehensive plan for the development of the river, with the idea that the first dam built will be properly located.

In his study of the subject the speaker has reached the following conclusions:

1.—As shown in the paper, the water supply of the Colorado River and its tributaries is not sufficient to irrigate the lands susceptible of irrigation lying in or adjacent to the Basin in the United States and Mexico.

2.—There are in round numbers, 1 000 000 acres of land in Mexico, which may be irrigated with the waters of the Colorado River. If these lands are irrigated, it will be necessary to dedicate to the desert forever about 1 000 000 acres of irrigable land in the United States.

3.—Any development on the river that will increase the low-water flow before such increase can be used in the United States, will enlarge the irrigated area in Mexico.

4.—If the Boulder Dam is built as planned, and 600 000 h.p. is developed at the dam, the low-water flow will be increased far in excess of the quantity that can be used for irrigation in the United States for many years to come. Long before this time, it will have been put to beneficial use on approximately 1 000 000 acres of land in Mexico. Although it is physically possible for this water to be diverted at a later date in the United States, it is not conceivable that this Government would dry up 1 000 000 acres of land in Mexico without first entering into negotiations with the Mexican Government. The time to deal with Mexico is before and not after its lands have been placed under irrigation, before and not after a large dam is built in this country, which will result in the irrigation of such a large acreage in Mexico.

5.—In the distant future it may be desirable to obtain from the Colorado River a domestic water supply for the Coast cities of Southern California. There is an excellent dam site in the Lower Grand Canyon which could be used as a point of diversion for a gravity system. Although a long tunnel would be required, such a system might be the most economical. The plan for a gravity system should not be rejected by engineers who never have seen the dam site. The plan now being advocated calls for a pump lift of 1 200 ft., which requires the use of 200 000 h.p. With the Boulder Dam Project preventing the development of 300 000 h.p. (according to the author), and the domestic water supply project forever dedicating 200 000 h.p. to pumping, it must be apparent that these plans should be carefully studied before any money is made available for construction purposes.

6.—There is urgent need for flood control, and this problem should be solved as quickly as possible. For more than a year the speaker has recommended that a thorough investigation be made of the Mohave flood-control reservoir site. As a result of his preliminary studies, he has reached the following conclusions regarding this site:

(*a*) It is the lowest known site on the river where adequate storage capacity can be obtained for flood control.

(*b*) It would surely form a unit of a comprehensive plan for the development of the whole river.

(*c*) It would probably cost less than any other storage site of equal capacity on the river.

(*d*) It would not destroy other valuable dam sites. The Boulder Dam, if built as planned, would destroy certain dam sites in the Lower Grand Canyon, and prevent the full development of the water resources of this section of the river.

(*e*) For the same storage capacity, a dam in Mohave Canyon would have about one-tenth the volume of masonry of a dam in Boulder Canyon.

(*f*) The Mohave Canyon site is accessible, being only 2½ miles from the railroad.

(*g*) Considerable quantities of sand and gravel, suitable for concrete, are available close at hand, and the granite in the abutment walls will provide excellent material for crushed rock.

(*h*) The depth to bed-rock is probably not great; the center pier of the Atchison, Topeka and Santa Fé Railway Bridge, 2½ miles above the dam site, is resting on bed-rock 70 ft. below the water surface.

(*i*) With the flood problem solved by a dam at Mohave Canyon, there will be ample time for studying the river and no necessity for proceeding except in conformity with its orderly economic development.

The speaker would make the following recommendations:

1.—A thorough investigation should be made of the Mohave Canyon flood-control reservoir site, including detailed surveys and diamond-drill borings. If suitable foundation is developed, a flood-control dam should be built at this site and operated in such a manner as to prevent any increase in the irrigated area in Mexico.

2.—While investigations are being made at the Mohave Canyon flood-control site, and during the construction of a flood-control dam, negotiations should be carried on with the Mexican Government with the idea of definitely fixing the quantity of water which is to be allotted to the lands in Mexico.

3.—Prior to a definite settlement with the Mexican Government, it would not be good business to permit the construction of a power dam on the Colorado River, which would increase the low-water flow of the river in excess of the quantity necessary to supply the land now under irrigation.

4.—While the flood-control dam is being built, and negotiations are being carried on with Mexico, the investigation of the Colorado River Basin should be continued, with the idea of evolving a comprehensive plan for the development of this river.

5.—If the resources of this great river system are to be developed in an orderly manner, a comprehensive plan of development must be made and agreed on by the Interior Department, the War Department, and the Federal Power Commission. Each development on the river should be a unit of this plan.

Arthur P. Davis,* Past-President, Am. Soc. C. E.—In this interesting paper the author presents some pertinent facts, and reaches important and far-reaching conclusions, some of which, if carried out, would result in the unnecessary destruction of large natural resources of land, water, and power. For this reason the speaker feels compelled to dissent from these conclusions and to give his reasons therefor.

The speaker is in accord with several of the author's maxims, as follows:

1.—No complete and inexorable plans for the development of the Colorado River should be adopted until more is known about the river, and the requirements of the future.

2.—No unnecessary evaporation should be involved.

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3.—No power head should be sacrificed unnecessarily.

4.—Agriculture should be given preference over power uses.

The plan proposed by the author violates all these maxims.

Importance of Storage.—The most outstanding and important feature of the Colorado River is the great fluctuation of its flow. The low-water years yield less than 9 000 000 acre-ft. below the canyons, and the high-water years yield about three times this quantity. The seasonal fluctuations are still more striking; autumn flows of 4 000 sec-ft. and less are common, and the high-water peak often reaches 150 000 and sometimes exceeds 200 000 sec-ft.

The low-water flow is already appropriated and used in irrigation, and an unusually dry season produces a shortage. Without regulation, the stream will produce only about 25% as much primary power as with proper regulation, and the effect of storage on irrigation is similarly important.

In addition, the great volume and long duration of the spring floods are such as to constitute an annual menace to all the valleys of the lower river, especially to the Imperial Valley, which, if entirely submerged, would be permanently destroyed, as it is largely below sea level.

These conditions make indispensable a large volume of storage capacity which fortunately Nature has made possible and feasible, with proper conservation and utilization. This, however, is not all.

Heavy Silt Content.—Nature sends down the river an annual average of about 100 000 acre-ft. of silt, which will fill to that amount any reservoir built within or below the canyon region. This silt clogs the channel, obstructs irrigation canals, and adds greatly to the flood menace and to the cost of irrigation operations. It makes the river unfit for municipal uses, for which it will be required extensively in the near future; all these conditions will be remedied by storage, as the silt will settle in the reservoirs, and the desilting of the river is one of the major requirements in changing it from a menace to a valuable asset. The average annual volume of this silt, however, is 160 000 000 cu. yd.—a greater amount than the total excavation performed by the Isthmian Canal Commission in digging the Panama Canal.

Desilting the river by settlement in reservoirs, fills those reservoirs to the extent of the volume of the silt deposited. Obviously, the time will come when all the reservoir space will be occupied by silt, unless some means of removal is applied. Any known method of accomplishing this is troublesome and expensive—many times as expensive as constructing additional reservoir capacity, as long as other good reservoir sites are available. It thus becomes necessary to conserve reservoir space to the greatest practicable extent.

The amount of storage necessary to equalize the flow of the river through all the years would be enormous, and hardly practicable. To just what extent it may pay to hold the water of extraordinary floods for use in dry years is a question about which the best engineers may differ, but all will agree that hold-over storage to a large extent is necessary if the river is to be properly used. It is necessary to retain a large amount of this storage in the Lower Basin, where most of the flow can be intercepted, in order to desilt the river and to control and utilize the floods of the lower tributaries.

Herman Stabler, M. Am. Soc. C. E., estimates the storage capacity necessary to maintain reasonable uniform annual flow between 10 000 000 and 20 000 000 acre-ft. The U. S. Geological Survey places it at 18 000 000 acre-ft.,* which latter figure may be used for present purposes. To this must be added a large quantity for flood control, and another for the storage of silt, to prolong the usefulness of the reservoir. At the very least, 25 000 000 acre-ft. is necessary to solve the problems presented. A much greater quantity would be useful in saving the water of abundant years, but is, perhaps, not justified at present.

Storage in Upper Basin.—The author states that the reservoirs above the canyon region should be used to regulate the river for use below. This is impossible, because they have not sufficient capacity, and are situated so that they intercept only a part of the flow, leaving a large area of water-shed unregulated.

The Ouray site, the largest of those listed in the paper, should be eliminated, as the Secretary of the Interior has promised in writing to dedicate this site to the construction of a railroad when required. The Kremmling site already has a railroad through it. This leaves as the principal known feasible sites in the Upper Basin, those given in Table 26.

TABLE 26.—Colorado River Reservoir Sites.

River	Reservoir.	Capacity, in acre-feet.	Average run-off, in acre-feet.	Cost.
Green	Flaming Gorge	3 120 000	2 300 000	$16 000 000
Yampa	Juniper	1 550 000	1 200 000	4 000 000
Colorado	Dewey	2 270 000	6 800 000	11 200 000
San Juan	Bluff	1 400 000	2 300 000	8 800 000
Total		8 340 000	12 600 000	$40 000 000

These reservoirs aggregate an insufficient capacity and would be relatively inefficient as two of them, if put to use, would seldom fill for lack of water, and a third is too small to control the drainage into it. Far more important, however, is the fact that these reservoir sites are needed for local use, for which they would not be a ailable if operated to accommodate the needs of the Lower Basin. This program would be an economic blunder of the first magnitude.

Their use for the needs of the Upper Basin may be of incidental benefit to the Lower Basin, but to dedicate them to the needs of the Lower Basin would destroy their local usefulness, and could be justified only in part, even if storage opportunities were not available in the Lower Basin; but this is not the case. The opportunity for storage in the Lower Basin is much larger, cheaper, and more efficient than in the Upper Basin. Boulder Canyon Reservoir can be built to a capacity five times as great as all the reservoirs noted in Table 26 combined, and at much less cost per acre-foot. If desired it can be built to a smaller size, at much less unit cost than the upper reservoirs and, later, can be enlarged when the encroachment of silt deposits makes it necessary.

* *Water Supply Paper No. 395*, U. S. Geological Survey.

Evaporation Should Be Minimized.—The author objects to Boulder Canyon as a large reservoir, because of alleged excessive evaporation. Curiously enough he proposes to substitute the Mohave Reservoir which has much larger area per unit of storage exposed to evaporation.

The author's curves show the comparison given in Table 27.

TABLE 27.—Comparison of Mohave and Boulder Reservoirs for Similar Capacities.

Capacity, in acre-feet.	Surface Area, in Acres.	
	Mohave.	Boulder.
10 000 000	112 000	69 000
15 000 000	136 000	90 000

In addition to having an area about 60% greater for a given capacity, for the height proposed, the Mohave Reservoir is 100 miles farther south, 500 ft. lower in altitude, much shallower, so it will become more heated, and is on an open plain, where the winds would have freer sweep than in Boulder Canyon. All these factors would have a tendency to increase the relative evaporation at Mohave; thus, for a given capacity, Mohave would probably have more than 75% more evaporation than Boulder Canyon Reservoir, if devoted to similar uses. Certainly, this new favorite cannot qualify on the score of economy of water.

Colonel Kelly seeks to justify this waste, by claiming a very large evaporation from the Mohave Valley in its present state. It is true that this valley now overflows, and, of course, experiences evaporation losses in proportion to the overflowed area; but if it is reclaimed, as it should be, the water applied to it would be usefully consumed in irrigation, and not wasted like that evaporated from a reservoir. Water evaporated from Mohave Reservoir is as truly wasted as if lost from any other reservoir in the Basin, and such evaporation is much greater in proportion to storage capacity than from any other proposed reservoir in the entire Basin, on account of its relatively broad, shallow character. As far as loss of water is concerned, it is by far the poorest site proposed.

The author charges that the height of dam proposed by the Reclamation Service—605 ft.—would make impossible the development of an 80-ft. fall between its back-water and Diamond Creek. This is not the case, for the dam in Boulder Canyon might be built 80 ft. higher whenever the additional capacity is needed in the future. This would have the merit of concentrating the power development at one plant instead of two, and save 80 to 100 miles of transmission line to the Coast. This height of dam, backing the water to Diamond Creek, is one of the alternatives presented by the Reclamation Service. On this fallacious claim, the author has based most of his condemnation of the proposal of the Reclamation Service.

The fact is, the Reclamation Service presented five alternative heights of dam, one of which is nearly that which the speaker prefers, 570 ft., backing water to the dam site at Bridge Canyon and creating a capacity of about

28 500 000 acre-ft.; it would cost about $43 000 000, and develop nearly 600 000 continuous h.p., or 900 000 on a two-thirds load factor. Marketed at 4 mills per kw., it would amortize the investment in about 40 years, meanwhile taking care of flood control and irrigation.

The author states that the water supply of the Colorado is insufficient to irrigate the land available for irrigation, yet he objects to a large reservoir capacity on the ground that it would waste water by evaporation, and insists on providing for passing the surplus at the rate of 80 000 cu. ft. per sec., rather than storing and using it for power and irrigation.

It should be emphasized that a dam of the height here proposed will develop power at a lower unit cost than any dam of less elevation, so that the additional storage for irrigation and the additional security against floods cost nothing. It is certainly better to hold the floods by storage, even if part of the water does evaporate, than to let them run to the sea, and fight them en route as demanded by Colonel Kelly.

Power Market.—Concerning the market for power, the author says: "It it not extravagant to conclude that all the power in the Canyon Section of the Colorado may be absorbed in the next 25 or 30 years." This is a higher rate of demand than was assumed in the estimates of the Reclamation Service, but evidently it is justified.

The Southern California Edison Company has repeatedly expressed willingness to expend from $30 000 000 to $40 000 000 per year to develop power in the Canyon if given the privileges.

The City of Los Angeles offers to take all the power allocated to it from the Boulder Canyon Dam after States and other municipalities are satisfied, and the electors recently voted by a large majority to authorize the City Council to negotiate such a contract. The city is certainly a responsible customer, and with all this evidence there need be no doubt that a ready market will be found for the power.

If the storage capacity allocated to flood control is too great, as implied by the author, a part of it can be used for power head and a part for storage, and the duty and consequent revenues of the reservoir increased.

Mohave Reservoir Wasteful and Destructive.—The alternative reservoir strongly advocated by the author is at Mohave Canyon. Not only is the Mohave Reservoir extremely wasteful of water by evaporation, but will require the removal of about 20 miles of double track on the main line of the Santa Fé Railroad, increase its length about 3 miles, destroy the great bridge at Topock, submerge the large railroad hotel, roundhouse, machine-shops, icing plant, 24 miles of yard tracks, and the entire City of Needles. It will also require the reconstruction of part of the Santa Fé Trail Highway. These damages have been included in the estimate, and the cost will be about as much as the dam itself. There is not included, however, the important fact that this reservoir would also submerge the whole Mohave Valley containing nearly 30 000 acres of rich, irrigable bottom-land for which there is an ample water supply when storage is provided, and that this water cannot be used elsewhere except in Mexico, because the regulated water supply is ample for all

lands feasible of irrigation in the United States, including the Mohave Valley lands.

Nature has provided two feasible dam sites that will utilize all the head available here, without submerging the Mohave Valley. One of these sites is above, at Bulls Head, and one below, near the mouth of Williams River. The latter is necessary for a high diversion dam for the Parker-Gila Valley Project, the construction of which the author assumes. It would be built to the greatest height possible without damage to the City of Needles, or about 100 ft. The proposed dam in Mohave Canyon would conflict with this, and would also submerge the dam site at Bulls Head, just above the Mohave Valley. It would leave undeveloped 40 ft. of power head below Boulder Canyon, which would thus be lost.

The dam site in the Mohave Canyon was examined twenty years ago and at intervals since by various engineers, including William Mulholland and J. B. Lippincott, Members, Am. Soc. C. E., and on account of the destruction of land, water, and other property, was unanimously regarded as infeasible, and for this reason, its foundations were not explored. Any money spent thereon would be wasted for reasons already given. It was never seriously advocated until it became necessary to kill the proposal to build a dam in Boulder Canyon and develop enough power to repay the cost. This emphasizes the real point at issue, that is, whether the immense resources of the Lower Colorado are to be retained in the control of the Federal Government or turned over to private corporations. This is a question on which engineers and others may honestly differ, and which the speaker will not discuss. It should be considered on its merits, and not combatted indirectly to the great sacrifice of the natural resources of land and water.

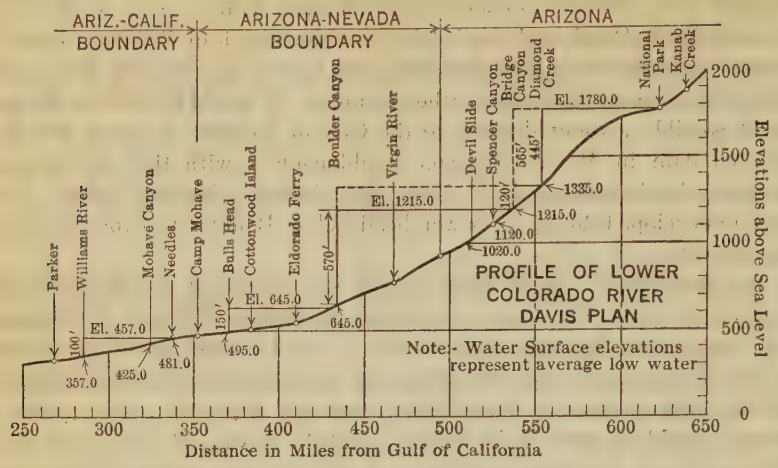
Wanton Destruction of Valuable Storage.—Perhaps the most destructive feature of the author's plan is the proposed building of a high dam at Devils Slide, thus restricting the height of the Boulder Dam to 387 ft., and its storage capacity to about 10 000 000 acre-ft. Such a reservoir would rapidly fill with sediment, and thus be destroyed much earlier than necessary.

It has been shown that large storage capacity is necessary to convert the Colorado River from a destructive torrent into a source of vast wealth, and that this storage will steadily be destroyed by silting. Yet, here is a proposal to build a high dam in the middle of the best reservoir site in the Basin, and thus wantonly destroy its value. It is idle to reply that other reservoirs can be provided above, because these are of limited capacity, and, therefore, of limited life; moreover, they do not intercept all the water nor all the silt.

Proper Plan of Development.—The plan of development proposed herein is shown on Fig. 21(*a*) as compared with that proposed by the author on Fig. 21(*b*).

The speaker's plan would be to build the Boulder Dam to a height sufficient to back up water to the dam site at Bridge Canyon. This dam would impound about 28 500 000 acre-ft. of water, control floods more perfectly than any smaller reservoir, and develop power more cheaply than any smaller dam, in sufficient quantity to repay its cost with interest.

Next, the Parker site should be improved to serve as a diversion dam for irrigation and municipal supply. With complete regulation at Boulder Canyon it will be a good power site.



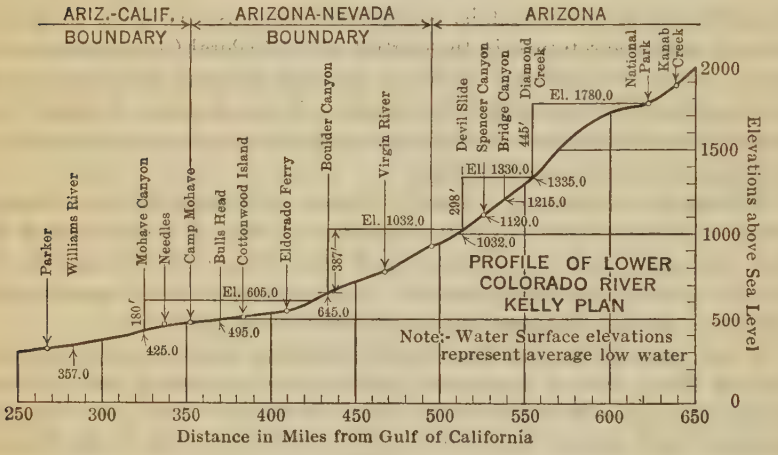


FIG. 21.

A dam at Bulls Head will back the water to the base of Boulder Dam, and with the regulation of that dam, will constitute a good power site. This leaves three alternatives for the development of the power head above Boulder Canyon, between Bridge Canyon and the National Park Boundary, as follows:

1.—If on that future day it is decided that more storage is necessary, the Boulder Dam can be raised 120 feet, its gross capacity increased to 48 000 000 acre-ft., less the accumulated silt, and the water backed to Diamond Creek, where a dam would be built to back water to the National Park.

2.—If it is then definitely decided that no more storage will ever be needed a dam can be built at Bridge Canyon to back water to the Park Boundary.

3.—If the decision is that additional storage may be needed some day, the dam at Diamond Creek can be built, and the space between this and Boulder Reservoir can be left open for still further consideration, that is, whether to build Boulder Dam higher, or to build a dam 128 ft. high at Bridge Canyon.

In any event, the construction of the dam at Devils Slide, as proposed, would be a serious economic blunder, as it would destroy the best reservoir site in the Basin, and seriously limit the usefulness of the river.

The plan proposed herein would utilize every foot of head between Parker and the National Park, except 38 ft. in the Mohave Valley, which would thus be preserved for agriculture, complying with the fourth maxim mentioned previously, as to the preference given to agriculture over power.

It complies with the first maxim, leaving several alternatives open for decision in the light of future information. It likewise complies with the other maxims. Contrasted with this, is the plan to issue corporate licenses for Diamond Creek, Devils Slide, and Boulder Canyon, and build a Government dam at Mohave Canyon, as proposed by the author, and shown on Fig. 21(*b*) for comparison. The two plans would have the results given in Table 28.

TABLE 28.—Comparison of Plans.

Site.	Head, in feet.	Surface area, in acres.	Capacity, in acre-feet.
	Davis' Plan.		
Williams	100	90 000	2 900 000
Bulls Head	150	20 000	1 600 000
Boulder	570	138 000	28 500 000
Bridge Canyon	570	16 000	3 000 000
Total	1 390	264 000	36 000 000
	Author's Plan.		
Mohave	180	130 000	13 000 000
Boulder	387	69 000	10 000 000
Devils Slide	298	4 000	400 000
Diamond	445	13 000	1 900 000
Total	1 310	216 000	25 300 000

It will be noted that the plan here proposed develops 80 ft. more head, and uses this head far more efficiently than the author's plan because its regulation is both greater and higher up the stream. Its power output would be greater by several hundred thousand horse-power.

The evaporation surface by the latter plan would be greater by about 22%, but owing to differing conditions already enumerated, the evaporation would be only slightly greater and the storage afforded would be 10 700 000 acre-ft. greater, and would endure more than 100 years longer. It does not submerge Mohave Valley, as proposed in the author's plan. In every respect, therefore, it is better.

A further comparison may be made by omitting the dam near Williams from Table 28. This produces the exhibit shown in Table 29.

TABLE 29.—Comparison of Plans—Williams Reservoir Omitted.

	Head, in feet.	Surface area, in acres.	Capacity, in acres.
Davis' plan	1 290	174 000	83 100 000
Author's plan	1 310	220 000	25 800 000

By this comparison over the same stretch of the river, the author's plan develops about 1½% more head, but owing to better regulation, higher on the stream, the speaker's plan would develop far more power. The latter would expose 46 000 less acres to evaporation, in a region where the rate of evaporation would be less, and afford more than 7 000 000 additional acre-feet of storage capacity, and, hence, would last 70 years longer; and it would not submerge the fertile Mohave Valley. It is clear, therefore, that the plan proposed by the author does not conform to the best development of the river and that it violates each of the maxims enunciated.

For irrigation and flood control, the Mohave site has no advantages over the Boulder site, as there are no tributary streams between, but the Boulder Reservoir would protect the Mohave Valley from floods, provide it with irrigation storage, and facilitate its reclamation, while a dam in Mohave Canyon would submerge and destroy that valley forever.

Storage in Boulder Reservoir would be beneficial for power development at Boulder Canyon, Bulls Head, and Williams, a total fall of 820 ft., while that at Mohave could be of no benefit for power except at its own site, which is so unfavorable that none of the power companies has applied for it. Storage at Mohave, therefore, would require duplication somewhere above, for power development.

Poor Rock in Glen Canyon.—The author expresses disagreement with the Consulting Board that examined the dam site above Lees Ferry in Glen Canyon and advised limiting the pressure on the rock to 20 tons per sq. ft. on account of its soft, friable character.

This Board included four experienced engineers all of whom have borne the responsibilities of designing and building some of the world's greatest dams, including the Don Pedro, Arrowrock, and Roosevelt Dams, and an eminent geologist, who made a very careful examination of the site and the material. It seems rather bold for one who has made no such examination to overrule them summarily on the strength of three samples of drill cores. Any one experienced in diamond drilling knows the persistent tendency of the core fragments to grind each other to powder in the rapidly revolving core-barrel, so that the soft parts are washed out and only the hardest survive. So great is this tendency, that the recovery of 100% of core even in the hardest rock is rare.

The rock in Glen Canyon is so soft that it is difficult to break off a sample without crushing it; and such a sample carried in one's luggage for a few days is apt to yield more fine sand than rock. The caution of the Consulting Board was clearly well founded.

Increase of Low-Water Flow.—The author opposes the increase of the low-water flow of the Colorado. He says:

"The Boulder Project would provide water in the Lower Basin at all seasons far in excess of present irrigation requirements. This water will pass into Mexico and there be used for irrigation. Once used, its withdrawal for use in the United States will be difficult, if not impossible."

The opposition to development is entirely unfounded. The proponents of the Boulder Project suggested the insertion of a clause in the bill reserving the benefits of storage to the United States. Legally and morally such a reservation is not necessary, but it would be good policy. Having provided the storage facilities, the United States would have both legal and moral rights to its benefits. The American lands being higher on the stream, these rights are easy of enforcement.

On the other hand, the regulated water being subject to appropriation on American lands would facilitate development in Arizona and California, and the feasible irrigation projects in those States would just about absorb those waters as they come from the power plants. To irrigate additional lands, as those in Mexico, it would be necessary to re-regulate the waters, which would be entirely in the hands of the American authorities.

Time of Construction.—The author states that flood control can be obtained more quickly by a dam in Mohave Canyon, than by one in Boulder Canyon, and compares the estimated time of constructing a large reservoir in Boulder Canyon with a much smaller one in Mohave Canyon. The investigations of Boulder Canyon have occupied nearly three years, and Colonel Kelly states that they are not sufficient. No investigations of Mohave Canyon have been made, and after these are completed, it would be necessary to take up negotiations with the railroad and hotel companies and hundreds of property owners for the removal of the railroad and the City of Needles. Such measures afford endless opportunity for delay whether carried out under private negotiation or by condemnation proceedings.

No such difficulties are found at Boulder Canyon, and it is believed that a reservoir of any given capacity can be built there more quickly than anywhere else, if all reasonable precautions are observed.

The Upper Basin's Interest in Boulder Canyon.—Four times in recent years, beginning in 1915, the irrigators of Imperial Valley have required and have taken from the Colorado River all the water flowing therein; each time it was less than the actual needs and a shortage occurred. As development progresses, this condition will be more frequent and intense, and if with present development, another series of low-water years should happen, such as occurred before and after 1902, the water shortage would be prolonged and disastrous.

Under the principles of the Supreme Court decision in the Wyoming-Colorado Case, the farmers of Imperial Valley would have the right to cause to

be closed during the period of shortage the head-gates of all canals above them, that are junior to their appropriations of 1900 and earlier. This would include all three of the Government projects in the Upper Basin, two in Colorado and one in Utah, and many private enterprises as well which have hard enough struggle without adding water shortage and litigation.

The irrigators in the Lower Basin do not desire litigation, as is shown by their avoidance of it during the past shortages. If a large reservoir is built and the water regulated, there will be plenty of water for them so there will be no litigation. The representative from Imperial Valley has testified that the District is willing to yield all claim to the low-water flow in exchange for a right in the reservoir, and no doubt the other irrigators served from the reservoir would do the same.

Without storage in the Lower Basin, the junior rights in the Upper Basin are in continual jeopardy, and the situation must become more and more a handicap to the financing of new irrigation enterprises in the Upper Basin. A large reservoir at Boulder Canyon will correct all this, and make water rights above secure from molestation. A small reservoir would not have this effect.

The author proceeds throughout the paper on the theory that a reservoir of, say, 4 000 000 acre-ft. capacity, would be as efficient for flood control as an equal capacity on the top of a reservoir six or eight times as large. This is not the case.

If a reservoir with a capacity of 25 000 000 or 30 000 000 acre-ft. were provided to regulate the river, it would, by constant use and discharge of the stored waters, enter the flood season with its storage depleted to an extent exceeding 8 000 000 acre-ft., and any capacity held especially for flood control would be added to this. Therefore, the available storage for controlling floods would be several times the total capacity of the small reservoir proposed by the author, and its efficiency correspondingly multiplied.

It is conceivable and possible to provide a reservoir at Boulder Canyon of such magnitude that the stream would be equalized through a series of years to a discharge of about 20 000 cu. ft. per sec. to be used for the development of power or some other beneficial purpose. Although such complete control might not be justified, any large reservoir approaching a capacity of 30 000 000 acre-ft., would approximate this result, and assure beginning the flood season in April with more than 10 000 000 acre-ft. of available space at the top of the reservoir to receive and regulate the summer floods. Any surplus received could be discharged during the flood season or soon after and not carried on to February, as alleged by the author.

Colonel Kelly ignores the great fundamental fact that the larger the reservoir provided, the more complete will be the control of floods and the more water will be available for power and irrigation; and the larger the reservoir, up to 35 000 000 acre-ft., the cheaper the unit cost of power developed, so that the added value for other purposes costs nothing.

He also ignores the fundamental fact that a deep reservoir of a given capacity with a moderate surface area will lose less water by evaporation and furnish more head for power than a broad shallow reservoir, which although it might require only a small amount of masonry for its dam,

requires a vast outlay for damages to cities, railroads, and other property, and costs about the same per acre-foot stored, besides destroying a great alluvial valley susceptible of high development.

The author deserves thanks for the frank official declaration of the settled policy of the Federal Power Commission, in his statement that:

"All the development needed on the Colorado will be built by private capital under adequate Federal and State regulation if the river is given over to development under the Federal Water Power Act."

This declaration is in the face of the mandatory provision of the law that the Commission shall give preference to applications from municipalities, such as are on file for the privileges of the Colorado power sites. That this defiance of law is a settled policy of the Power Commission is an important fact in considering legislation affecting not only the Colorado, but any other stream under its jurisdiction.

If the author really believes his statement that the Mohave is a good power site, and should be the first development, why does he not require the power companies to begin their development by building the Mohave Reservoir? None of the power companies has applied for this privilege, and they would probably spurn it, as the results would not pay for the damages caused to railroads and other property. The mere mention of such a proposition should be considered absurd, as it really is.

C. E. Grunsky,* Past-President, Am. Soc. C. E.—The Colorado River presents one of the nation's largest and most pressing domestic problems—a problem which, however, has its international aspects, that, through inaction on the part of the Federal Government, are annually growing more serious. As Adviser to the Secretary of the Interior in 1907, the speaker called attention to the anomalous situation which had resulted from the operations of the California Development Company, whose canal located in part in Mexico was supplying irrigation water from the Colorado River to the lands in Imperial Valley, California, and suggested a plan of procedure to prepare the way for a comprehensive treatment of the river problem which would be advantageous both to the United States and to Mexico. Nothing came of the suggestion, however.

The problem cannot be solved without giving consideration to the fact that the Colorado is an international river. The interstate pact formulated by a Commission headed by Herbert Hoover, Hon. M. Am. Soc. C. E., now ratified by all the interested States, except Arizona, will be, when consummated, only a first step toward the solution of this problem.

If State boundaries are to be considered and conflicting interests between upper and lower groups of States are to be recognized, as indicated in the pending pact, then, too, the States within the two groups may well ask, as does Arizona, "What will be the situation 40 or 50 years from now?" Each of the States in the upper group wants to know what proportion of the water which is to be left in the river for use of the lower group of States it will be expected to furnish—just as each of the States of the lower group may desire

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

to know at the outset what proportion of the water and what proportion of the power output will ultimately fall to its share.

It is not surprising, therefore, to find Arizona with only a limited area that can be brought under canal at reasonable cost, in the attitude of inquiry. There is some basis for the fear, for example, that if a dam is constructed at or near Boulder Canyon and a power plant is placed in operation, the predominating benefit will go to those regions where industries are already well established—that the southern part of California will absorb an undue proportion of the energy made available, at the cost of Arizona's future. A similar fear is expressed in the matter of the use of water for irrigation. Who, if there be no understanding on this point in advance, shall prevent the extension of irrigation to the most favorably located lands regardless of State, or even of National, boundaries, thereby possibly depriving one State or another of opportunities within its boundaries which, although perhaps not immediately attractive, may yet be of equal or greater value in a broad plan based on the ultimate greatest good to the greatest number.

No reasonable regulation of the flow of the Colorado River by storage appears to be feasible except with the approval of and under the control of some higher authority than that of the individual State. This principle has already been recognized in the creation of the Commission which formulated the interstate pact. The Federal Government is the agency which, logically, should effect the regulation and apportion the output. As a possible alternative, the suggestion of Governor Scrugham, of Nevada, in an address at a meeting of the Pacific Coast Electrical Association, deserves consideration. He suggested that an interstate power and irrigation district be formed to plan and carry out the Colorado River project. If those who are directly concerned prefer district to Federal control, why should they not have it, provided, always, that the international phases of the problem are not overlooked?

An interstate district, however, any more than the United States, could not prevent development along lines of least resistance, regardless of State boundaries, unless there be some pre-arranged plan of development, with power and water allotments adjusted in some measure to political sub-divisions. The time element should be taken into account and the desirable ultimate conditions should be forecast and carefully weighed. Although the ideal solution of such a problem as that of the Colorado River might ignore State boundaries, it must be submitted that their recognition may aid in securing desirable widespread, rather than concentrated, utilization of both water and power.

In Mexico extensive areas have already been brought under irrigation with water from the Imperial Canal. There is satisfaction in the knowledge that the water in the Colorado River originates in the United States and that were it all retained and used within these borders, if possible, the nation's neighbor to the south would have to be content with a protest. This will never happen. Thus far, however, there has been no limit set to the quantity of water which is ultimately to be allowed to flow into Mexico, and the irrigated area in Mexico has been growing apace. On this subject the United States Govern-

ment was warned, as already stated, in 1907, at a time when Colorado River water was used in Lower California on less than 10 000 acres. At present, the irrigated area in Lower California is about 200 000 acres. What shall be the limit and what agency other than the United States can fix the limit?

There is plenty of land in Mexico to use the water—about 800 000 acres in the river delta and, on the mesas of Sonora, an additional 500 000 acres or more. The ultimate extent of irrigation in Mexico need not then be regarded as limited by the physical features of the country, but solely by the quantity of water temporarily and ultimately available.

It should be noted in this connection that the engineers of the U. S. Reclamation Service, who have given much thought and study to the Colorado River problems, report that, after all the lands in each State of the upper group that can be supplied with water from the Colorado River at reasonable cost have been brought under irrigation, there will still be an ample supply, not only to meet the compact requirement of the lower group of States, but also for a fair delivery of water to Mexico.

Again, the Colorado River menaces certain areas in the United States and other areas in Mexico, whenever at flood stages it runs wild in Mexico. The great menace is to the lands which lie below sea-level, partly in Mexico, but mainly in the United States. The places where this flood menace must be fought are in Mexico. To be sure the frequency of floods can be reduced materially, and channel deterioration by silt deposits can be decreased materially with such works as the contemplated Boulder Reservoir, but occasional floods originate in parts of the river's water-shed below Boulder Canyon, and even after the construction of a Boulder Reservoir or its equivalent, these floods would still remain a problem feature in the river delta south of the boundary. The fact should be recalled, in this connection, that the discharge of the Gila alone may bring the Colorado River at Yuma to a full flood stage as it did in November, 1905, when the river broke from its channel and took a course into Salton Basin.

The flood control problem, therefore, involves not only the retention of the flood waters of the upper river in reservoirs, but also a proper treatment of the lower river to care for the high stages that will still occur occasionally after up-river reservoirs have been built.

The United States should arrange all matters pertaining to flood control, not only in so far as the construction and operation of storage reservoirs are concerned, whether these be built by an interstate district, by a single State, by a group of States, by a private corporation, or by the United States, but also in the matter of providing a direct permanent channel for the Colorado River from the southern boundary of California to the Gulf.

In other words the United States must maintain itself in a position to keep the flood menace for all time at a minimum and to assure respect for interstate and international obligations. When the United States, however, either by direct action or through selected agencies, reduces the flood menace and conserves the waste water of the river for use in generating power and for irrigation, the resulting benefits, both in this country and in Mexico, should

be ascertained and the cost of the works, or better, the cost of the service rendered, should be charged in some equitable manner to those who use the output of power and water and to those who benefit by the flood-control works.

Surely there should be no difficulty in reaching an understanding on this point with Mexico, neither should there be any difficulty in agreeing with Mexico on the project for flood control within that country, nor yet as to the limit beyond which a delivery of Colorado River water to Mexico cannot be assured.

F. E. Weymouth,* M. Am. Soc. C. E. (by letter).—The writer would like to point out discrepancies in data which, in the paper, have been credited to the U. S. Bureau of Reclamation. For example, Table 11 does not appear in any report of the Bureau of Reclamation, as far as the writer can learn; the equivalent information as it appears in a report which he made in February, 1924, is as given in Table 30.

TABLE 30.—Irrigable Areas and Water Requirements.
(Areas, in Thousands of Acres; Consumption of Water, in Thousand Acre-Feet.)

	Present:		Immediate Future.		Near Future.		Distant Future.		Total.	
	Area.	Water.	Area.	Water.	Area.	Water.	Area.	Water.	Area.	Water.
Above Boulder.....	*	*	773	1 349	1 063	1 958	989	1 879	2 825	5 186
Between Boulder and Laguna.....	40	120	166	498	47	141	216	937	469	1 696
Below Laguna in United States....	469	2 111	430	1 935	.....	.....	.....	.....	899	4 046
Total for United States.....	509	2 231	1 369	3 782	1 110	2 099	1 205	2 816	4 153	10 928
In Mexico.....	190	855	300	1 350	310	1 395	†	.....	800	3 600
Grand total.....	699	3 086	1 669	5 132	1 420	3 494	1 205	2 816	4 993	14 528

* Not shown, as Table 30 shows only anticipated future demands on present stream flow.
† 200 000 acres not dependent on Colorado River discharge is not included.

A part of Table 19 is designated "U. S. Bureau of Reclamation Plan", and contains in its heading an assumption of up-river storage development, which, to the writer's knowledge, no report made by the Bureau of Reclamation has ever embodied. Even if this heading were changed, the data presented as to power head, evaporation, irrigation storage, and flood-control storage, do not conform to similar data presented in that report covering development to Elevation 1 250, which Table 19 is probably intended to present. Further, this table contains no reference to the alternative plan presented at the same time, contemplating development to the elevation of the Bridge Canyon site. These discrepancies should be borne in mind in considering the discussion following Table 19.

* Pres., Brock & Weymouth, Inc., Philadelphia, Pa.

Investigations of the Colorado River Basin were started by the Reclamation Service in 1904 with the view of augmenting the water supply for irrigation in the Lower Valley and of controlling the ever-present flood menace in the Delta region. Extended investigation of the Upper Basin indicated a lack of the requisite storage at reasonable cost, therefore, studies of storage sites in the lower river were undertaken. After a preliminary study of the problem and a reconnaissance of the river below the mouth of the Virgin River, work was concentrated, in 1919 and thereafter, on the better dam sites in Boulder and Black Canyons.

The primary object was the regulation of the river for irrigation and flood control. As these investigations progressed, it became evident that a large amount of power could be developed without interfering with the primary use of the reservoir for irrigation and flood control, and that at the dam sites considered it would be possible to develop power which could be sold at a price sufficient to repay the cost of construction. In all these studies, the power problem was considered only as incidental to that of irrigation and flood control. Studies were largely concentrated at the Boulder Canyon Reservoir site where the requisite storage could be obtained at the lowest cost and where the development could be made without interfering with other uses of the river.

A preliminary report on this subject under the title, "Problems of the Imperial Valley and Vicinity," was made in 1922 by Arthur P. Davis, Past-President Am. Soc. C. E., then Director of the Reclamation Service. The studies were continued after that report was completed and the results embodied in a voluminous report, containing nearly 2 000 typewritten pages, submitted by the writer in February, 1924, under the title, "Report on the Problems of the Colorado River Basin." When this report was written, the data available on dam sites above Boulder Canyon were practically limited to the Glen Canyon and Diamond Creek sites.

In 1923, the U. S. Geological Survey made some topographical surveys and a general geological examination of the Colorado River Gorge from Lees Ferry to Las Vegas Wash, the latter being between the Boulder and Black Canyon Dam sites, which were investigated by the Bureau of Reclamation for the Boulder Canyon Reservoir site. A few days prior to the completion of the writer's report, the Geological Survey furnished an approximate elevation of some of the dam sites below Diamond Creek. Brief studies, therefore, were made in the time remaining and led to the submission of an alternative plan limiting the height of the Boulder Canyon Dam to the level of the Bridge Canyon Dam site. It was pointed out that the success of this plan would depend on the feasibility of the Bridge Canyon for a high dam and power plant.

Since then the Geological Survey has provided a preliminary profile of the Colorado River through the stretch investigated in 1923, as well as the topography of the principal dam sites noted by the survey party below Grand Canyon National Park. These data, in addition to those obtained from investigations by the Bureau of Reclamation, have made it possible to extend the previous studies and to determine the best plan for the development of

the lower river, having in mind at all times the paramount importance of irrigation and flood control. Some of the results of these studies are embodied in this discussion. The section of the river through the Park has been disregarded, because of public aversion to its development, while the part above the Park is so far distant from large power markets that its development will not be economical until the entire power possibilities below the Park become fully utilized.

The more important factors involved in the proper development of the entire section below the Park in the order of importance are, as follows:

1.—Adequate storage capacity for irrigation and flood control with a minimum of evaporation loss.

2.—Construction cost.

3.—Maximum power output with maximum flexibility.

4.—Minimum destruction of developed and undeveloped property.

Adequate Storage Capacity for Irrigation and Flood Control.—This factor is by far the most important, as the use of water for irrigation must take precedence over that for power in the Colorado River Basin. The object of the development should be first, the protection of present irrigated lands from the ever-present dangers of inundation and water shortage and thereafter the provision of adequate storage capacity to permit the utmost development of the water resources of the Colorado River Basin. Other waters cannot be substituted to irrigate lands in the Colorado River Basin, but other sources can provide power.

Under present conditions less than 1 000 000 acre-ft. of active storage are required to insure an adequate irrigation supply for the Lower Colorado areas. With further development throughout the Basin, this volume will soon increase rapidly. In the writer's report of February, 1924, it was estimated that 25 000 000 acre-ft. would ultimately be required for this purpose alone if up-stream developments did not create material hold-over storage. Undoubtedly, considerable storage of this type will materialize, but it is believed that 15 000 000 acre-ft. of storage will be required for this purpose on the lower river.

The present silt inflow at Boulder Canyon is estimated at 80 000 acre-ft. annually. It will gradually decrease as up-stream reservoirs are constructed. To prevent the encroachment by silt on the storage needed for irrigation purposes for a long period of years, an additional capacity of 5 000 000 acre-ft. is needed.

Floods should be limited to not more than 40 000 sec-ft. at Yuma, as discharges of more than 50 000 sec-ft. are difficult of control and seriously endanger the present levees. The uncertainties attending any change in the regimen of a stream like the Colorado River dictate that ample provision should be made to meet contingencies. Under present conditions, such control will require 8 000 000 acre-ft. of storage, decreasing to 5 000 000 acre-ft. when the up-stream development becomes considerable.

From the foregoing it will be seen that a total capacity of 25 000 000 acre-ft. should be provided to meet future conditions. That such a capacity

is not needed to care for irrigation, flood control, and silting in the immediate future is apparent. The provision of this or even a greater capacity at the present time is not an economic blunder, however, as the full control of present flow for the prevention of waste and, therefore, the maximum power output requires a storage of more than 30 000 000 acre-ft.

The location of this storage is also an important factor. From the standpoint of flood control and avoidance of irrigation waste, storage should be as near the point of use as possible. To restrict evaporation losses, the reservoir should be deep and located in the Canyon Section.

Construction Cost.—Even with the cost of the dams added to the power feature, power can be developed more cheaply at Boulder Canyon for the market available than at any other point on the Colorado River, or elsewhere. Modification of plans for purely storage development to include power development must provide power at a minimum cost per unit, in order that the additional investment may be justified and the burden on the power users reduced to a minimum. The more recent studies have assumed that the cost of storage for irrigation and flood control should be saddled on the power development.

Power Output.—Although adequate provision must be made for irrigation and flood control, any accepted plan must offer no avoidable interference with the maximum possible power development. Maximum power output is desirable as the presence of a market in the near future for all power that may be developed below the Park is generally conceded. As power depends on head and quantity of water, it follows that the maximum possible output would be obtained with stream control at the upper reaches of a section and utilization of the entire head below. As no feasible storage possibilities with sufficient capacity exist above the Boulder Canyon Reservoir site, it follows that, in order to obtain a maximum product, the plan adopted must be a compromise between a sacrifice of operating head and of storage capacity.

A considerable portion of the energy used in Southern California at the present time is devoted to irrigation pumping, which varies greatly from year to year with the stream flow. No doubt such pumping will continue to take a considerable part of all power marketed there. Some of the power systems with which the Colorado River development would be interconnected are less fortunate in storage possibilities. The best use of Colorado River power is possible only with a large storage reserve at strategic points, that will permit a flexible power output.

Destruction of Property Values.—The Mohave Valley Reservoir, if developed to Elevation 600, as proposed by Colonel Kelly, would require the reconstruction of about 26 miles of double-track main-line railroad (Santa Fé). The Town of Needles, Calif., with a population of more than 2 500, would have to be moved to a new site, together with extensive terminal facilities of the railroad company. Irrigable land, to the extent of 40 000 acres, much of it Indian reservation, would be submerged. A school would also have to be moved. Parker Valley Dam, if developed to Elevation 425, would not destroy

any property of value. Developed to Elevation 458, the lower part of Mohave Valley would be submerged to the extent of possibly 10 000 acres.

Field Data Available for Designing Work.—

Diamond Creek Dam Site.—The topography and drilling data were obtained from Mr. J. B. Girand, applicant before the Federal Power Commission for this site.

Bridge Canyon, Spencer Canyon, and Devils Slide Dam Sites.—The topography at the dam sites was obtained in 1923 by the U. S. Geological Survey, but in most cases in insufficient amount to cover the sites for power houses as well. No data are available on foundations. The estimates of cost are based on assumed depths to rock of 60, 90, and 120 ft., the middle depth being used for a general comparison of plans. The accessibility of the sites has been assumed without actual knowledge of this condition.

Boulder Canyon Reservoir Site.—Detailed topography of the reservoir area and dam sites has been obtained; surveys made for connecting railroads indicate a readily accessible site. The two most favorable dam sites indicated by detailed geological investigations have been thoroughly drilled. Concrete materials have been located and subjected to practical tests to determine their suitability with gratifying results. This work was carried on from 1918 to 1923.

Bulls Head Reservoir Site.—The topography for this site was obtained in 1902 and 1903. The location was diamond-drilled in 1903 by the U. S. Reclamation Service as reported in the Second Annual Report of that Service.

Mohave Valley Reservoir Site.—The general topography of this reservoir site was made by the Reclamation Service in 1902 and 1903 and that of the dam site by the U. S. Geological Survey in 1923. No drilling has been done. Estimates of the depth to rock are based on conditions existing at the railroad bridge a few miles above and at other dam sites on the river. The cost of necessary changes in railroad trackage and facilities is computed from information furnished by the Railroad Company.

Parker Dam Site.—The general topography was taken in 1902-03. Additional topography was furnished by Fred A. Noetzli, Assoc. M. Am. Soc. C. E. The site has not been tested for foundations.

Designs and Estimates.—Designs and estimates for the Black Canyon Dam have been worked out in great detail for various heights, based on complete field data.

Information on the Diamond Creek, Bulls Head, and Parker sites is in such shape that the designs and estimates can be considered little better than preliminary.

The foundation conditions and right-of-way difficulties at the Mohave Valley site are unknown. There are so many uncertainties connected with the cost of this development that any estimates made with available data may be misleading. Under no circumstances should this site be developed.

The Bridge Canyon, Spencer Canyon, and Devils Slide sites have only incomplete topography from which estimates can be made.

Although an infinite number of plans for the development of the Grand Canyon-Parker Section may be advanced on the basis of known and assumed

dam sites, it is believed that the layouts presented in Figs. 22 and 23 cover all the more favorable plans.

TABLE 31.—Statistics of Plans for Colorado River Development.

Plan.	Conditions with Present-Day Water Supply.				Conditions with Full Up-Stream Development and Complete Up-Stream Storage Control.				Total storage capacity for silt, floods, and equating stream flow, in acre-feet.
	Total head utilized in feet.	Total horse-power available.	Total cost.	Cost per installed horse-power.	Total horse-power available.	Total cost.	Cost per installed horse-power.	Average evaporation loss, in second-feet.	
A	1 353	3 080 000	$339 000 000	$110	2 810 000	$316 000 000	$111	1 400	32 700 000
B	1 283	3 155 000	359 000 000	113	2 620 000	312 000 000	118	1 550	38 900 000
C	1 338	2 665 000	331 000 000	122	2 810 000	336 000 000	119	1 100	20 700 000
D	1 338	2 350 000	303 000 000	128	2 825 000	331 000 000	116	940	14 900 000
E	1 352	3 280 000	388 000 000	116	2 900 000	341 000 000	117	2 110	41 600 000
F	1 282	3 317 000	398 000 000	119	2 715 000	335 000 000	123	2 260	47 800 000
G	1 337	2 967 000	370 000 000	126	2 905 000	354 000 000	121	1 810	29 600 000
H	1 337	2 770 000	356 000 000	126	2 920 000	356 000 000	121	1 650	23 800 000

In comparing these plans as regards their most important features (Tables 31 and 32), two conditions of water supply have been assumed, namely, present supply without up-stream control and possible future supply with stream flow fully equated. Neither condition will ever fully materialize. By the time the entire river below the Park can be put to power use, much irrigation and power development may take place above. Full control of the river by up-stream storage is impracticable. Actual conditions will gradually leave the "present" status and approach that of full up-stream development.

TABLE 32.—Comparison of Plans for Colorado River Development.

Conditions with Present-Day Water Supply.						Conditions with Full Up-Stream Development and Complete Up-Stream Storage Control.									
Plan.	Total horse-power available.	Plan.	Total cost.	Plan.	Cost per installed horse-power.*	Plan.	Total horse-power available.†	Plan.	Total cost.	Plan.	Cost per installed horse-power.	Plan.	Average evaporation loss, in second-feet.	Plan.	Total‡ storage capacity for silt, floods and equating stream flow, in acre-feet.
F	3 317 000	D	$303 000 000	A	$110	H	2 920 000	B	$312 000 000	A	$111	D	940	F	47 800 000
E	3 280 000	C	331 000 000	B	113	G	2 905 000	A	316 000 000	D	116	C	1 100	E	41 600 000
B	3 155 000	A	339 000 000	E	116	E	2 900 000	D	331 000 000	E	117	A	1 400	B	38 900 000
A	3 080 000	H	356 000 000	F	119	D	2 825 000	F	335 000 000	B	118	B	1 550	A	32 700 000
G	2 967 000	B	359 000 000	C	122	A	2 810 000	C	336 000 000	C	119	H	1 650	G	29 600 000
H	2 770 000	G	370 000 000	G	126	C	2 810 000	E	341 000 000	G	121	G	1 810	H	23 800 000
C	2 665 000	E	388 000 000	H	126	F	2 715 000	G	354 000 000	H	121	E	2 110	C	20 700 000
D	2 350 000	F	398 000 000	D	128	B	2 620 000	H	356 000 000	F	123	F	2 260	D	14 900 000

* Based on estimates for installations using convenient power units with total power approximating available power.

† Based on complete equation of stream flow by up-stream storage.

‡ Total capacity behind dams, except lower 67 ft. of Parker Dam.

The evaporation from the reservoirs is based on an estimated loss of 5 ft. per year over the mean exposed area in each case. This value may be con-

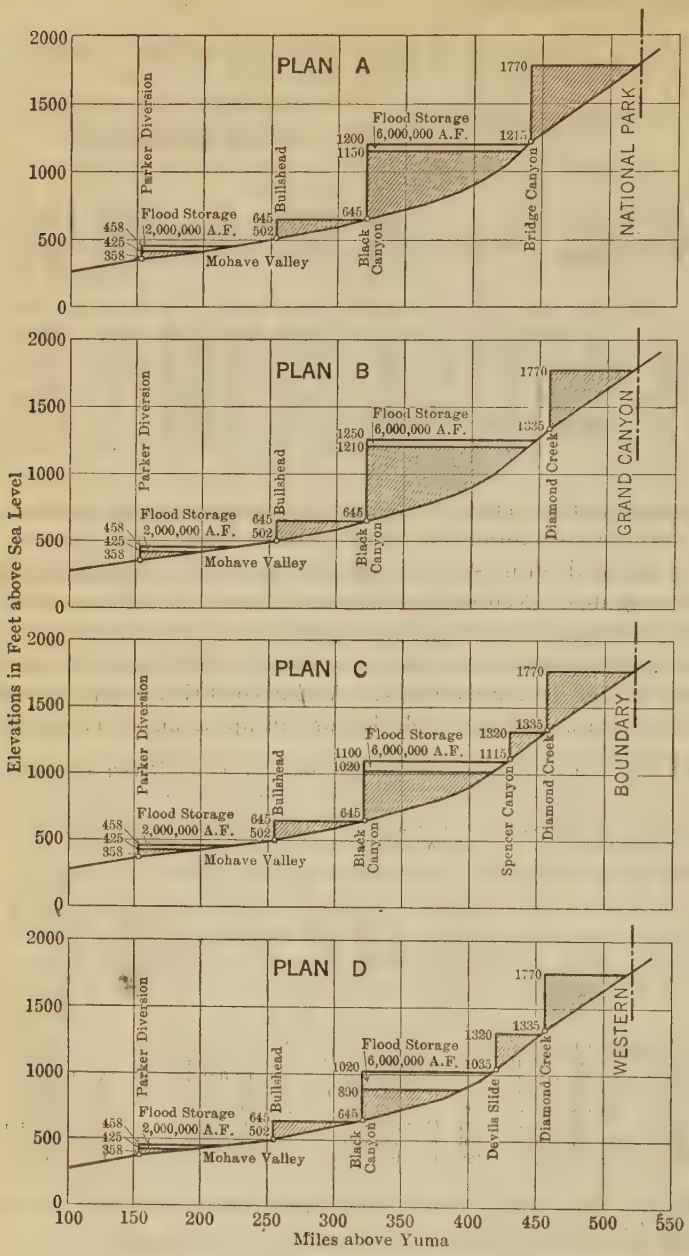


FIG. 22.—DEVELOPMENT OF COLORADO RIVER BELOW GRAND CANYON, PLANS A TO D, INCLUSIVE. FLOOD STORAGE INDICATED IS THAT REQUIRED UNDER PRESENT CONDITIONS.

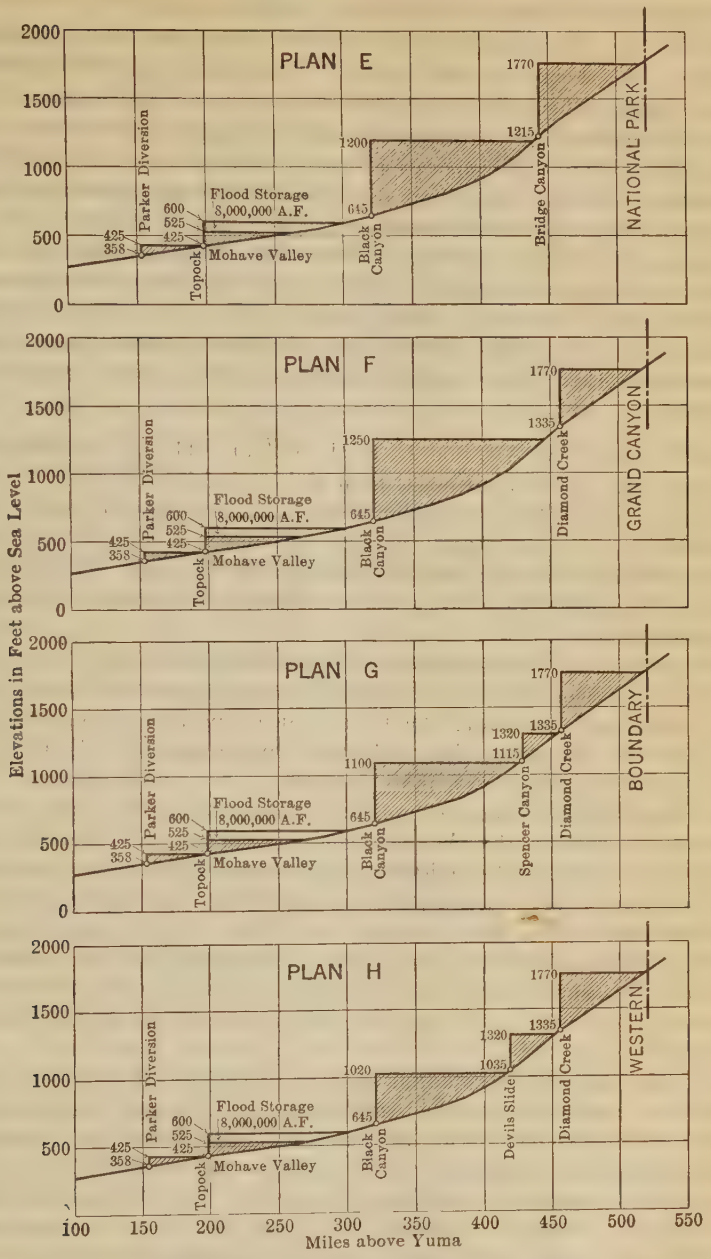


FIG. 23.—DEVELOPMENT OF COLORADO RIVER BELOW GRAND CANYON, PLANS E TO H, INCLUSIVE. FLOOD STORAGE INDICATED IS THAT REQUIRED UNDER PRESENT CONDITIONS.

sidered an average. Actual losses will be higher in the lower desert region below Boulder Canyon than in the Canyon Section proper.

Plan A, Fig. 22, exclusive of the Parker Dam, was first suggested by the writer in his February, 1924, report. That plan, with the Parker Dam included, presents the best scheme of development for the lower river as a whole. Its outstanding advantage is that it supplies the needed storage capacity with a minimum evaporation loss. It provides superior flood control by regulating Williams River, and it entails a minimum destruction of irrigable areas. The cost of construction is the lowest of all the plans, both under present and future conditions. With only partial control, such as will actually exist, the power output for these plans with small storage capacity would suffer far more severely than Plan A. The cost of the Parker Dam is included in the total cost of this plan. It would provide a much needed diversion for Parker Valley and for contemplated irrigation of the higher lands below.

G. E. P. Smith,* M. Am. Soc. C. E. (by letter).—There is no subject more timely, especially to the Southwest, than the one under discussion. Perhaps no other great undertaking, of immediate concern, involves so many engineering problems of a controversial nature, and none is more worthy of the assistance which results from open discussion by the Society. The author is to be thanked for presenting clearly the viewpoint of the staff of the Federal Power Commission.

Among those who have studied the problems of the Colorado, is the U. S. Reclamation Service, which has carried on investigations and studies of the highest importance, including the testing of many foundations, for twelve years. Likewise, the pioneering and other investigations of E. C. La Rue, M. Am. Soc. C. E., have been most valuable.

The State of Arizona has been somewhat laggard in this regard. However, it organized, in 1922, the Arizona Engineering Commission to make a reconnaissance study of the irrigation possibilities from the Colorado River within the State. This Commission, of which Mr. La Rue was Chairman, reported in July, 1923. The Legislature of 1923 made a special appropriation of $14 000 for hydrometric work on the Colorado for the biennium 1923-25; an initiated measure authorizing extensive studies of the Colorado and appropriating $100 000 for the purpose, was also voted on in the November, 1924, elections.

As a large part of the potential water power of the Colorado lies within Arizona, this State will exercise considerable control over the development program. Within its boundaries, the State has control over the appropriation of water for irrigation and other uses, and, also, has control over the stream and its bed in all matters not related to navigation. The Federal Government, by reason of its ownership of abutting lands, also can exercise control over the proposed developments, which control is exercised through the Federal Power Commission. The State and the Federal Governments, therefore, must cooperate in the control of the power development of the river. Inasmuch as

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about 90% of the power development in the canyon region will be within partial control of Arizona, it is desirable that the State should be represented in the studies and conferences which must precede any further construction on this important stream.

Water Supply.—It has been assumed that the extensive records of stream flow on the Colorado and its tributaries were adequate as a basis for study. However, Herman Stabler, M. Am. Soc. C. E., has added material of great value by extending the records backward more than 20 years, as given in Table 5. The net result of this study is to indicate that the safe water supply of the river may have been over-estimated by about 12 per cent. In studying the water resources of the Gila River a few years ago a somewhat similar change in flow was noted, that is, a greatly increased run-off during the past twenty years. The average rainfall at Tucson for the period, 1885-1904, was 10.25 in., whereas the average from 1905 to 1923, inclusive, was 12.91 in. The records of rainfall do not demonstrate any periodic cycle, but they do give warning that the discharge records since 1902 are not an entirely safe basis for planning the utilization of the river.

It has long been apparent that the water supply of the Colorado is inadequate for all the demands that will be made on it, as is the case with many other streams of the West. In the report of the Arizona Engineering Commission, Mr. La Rue pointed out that there is a 1 000 000-acre project with diversion point near Parker, Ariz., the development of which may be feasible in 20 or 30 years, but that there may be no water supply for it. Other projects may yet be found that are comparable with some of those now considered feasible.

The adequacy of the water supply in the seven States will depend in a measure on the quantity of water granted to Mexico through a treaty. Such agreement may be complicated by a situation the reverse of that on the Colorado, which exists on the Lower Rio Grande, where the Conchos River of Mexico is a big factor in the water supply of lands in Texas. The writer agrees with the author that a treaty with Mexico, covering the quantity of water to be delivered at the International Boundary, should be negotiated as soon as practicable. It should require, also, the payment of a part of the cost of storage by the holders of the Mexican lands, in proportion as they are permitted to share in the benefits of storage.

It must be confessed, however, that the use of water in Mexico may increase rapidly to more than the quantity agreed on by treaty, and the probable result would be another treaty granting a larger quantity.

Comparing Mr. Stabler's estimate of dependable supply with Table 11, it is apparent that ultimately the shortage may approximate 6 000 000 acre-ft.

In California, the City of Los Angeles has filed application for a diversion of 1 500 sec-ft. in the vicinity of Blythe for use in Los Angeles and vicinity. It would add to the value of the paper if the author would state whether allowance has been made in Table 11 for such a diversion.

The Division of the Water Supply.—As between the seven States that are interested in the use of the Colorado River for irrigation, the greatest care

should be exercised to take no irrevocable steps that will prevent the highest use of that portion of the water supply remaining to the United States. It is impossible to state at present just where and how the last residual unappropriated water should be used, or where it will be most needed.

Six of the seven States are so-called priority States, that is, States where the water laws are based on the doctrine of beneficial use according to priorities and where riparian rights do not exist. The seventh State, California, since the time limit for asserting riparian rights expired in December, 1923, is also a priority State, at least so far as the Colorado River is concerned. The decision of the Supreme Court in the Laramie River case is applicable, therefore, to the Colorado River Basin—in brief, the doctrine of beneficial use according to priorities is the basis for water division throughout the Basin, regardless of State boundary lines. There is no ownership or permanent control of the water by any project or any State. An appropriator acquires the right to use the water, as far as he can make beneficial use of it, and as long as he does use it. The building of a reservoir is not necessarily an appropriation; actual beneficial application of the water to land, without undue delay, is required also. So much has been said and written in support of this principle of water law and the need for it in States where irrigation is practiced, that it would be superfluous to enter into a discussion of it here.

The author states that the Boulder Project would put to beneficial use for power practically all the water in the Colorado River and would thereby acquire prior rights for it all. This suggests that a treaty between the seven States might be limited to an agreement that the use of water for power should of itself create no rights as against its later appropriation for irrigation, either up stream or down stream. It is an open question as to whether the author's statement is correct. A year ago (1923), the Federal Power Commission proposed to stipulate in the licenses for power that no rights prejudicial to the development of irrigation are acquired by the licensees. Many authorities believe that such a stipulation would protect the Upper Basin States, especially if similar provisions should be made in the power permits to be issued by the States. In the absence of an agreement, a test case should be brought before the United States Supreme Court, to determine whether a power dam can acquire rights as against irrigation. The Imperial Valley should bring such a suit against the permittees of the Flaming Gorge Dam site.

A more comprehensive contract or agreement may be needed between the three States of the Lower Basin and the Federal Government. It should include provisions covering (1) the location of any dam or dams on State boundary lines; (2) the subject of taxation; and, (3) the operation and maintenance of structures built on boundary lines, if built by public agencies, and the allocation of their costs and benefits.

Location of First Dam.—The writer agrees fully with the author that "storage for equalization of flow should be provided above the Canyon Section, and that storage for re-regulation of flow should be provided at the bottom of the Canyon Section"; and also that the storage capacity required

at the present time for flood protection and for insuring a late-summer irrigation supply is moderate in amount, say, 4 000 000 acre-ft. exclusive of silt storage. He does not agree as to the importance of locating the reservoir for flood protection as far down stream as practicable, and considers that the author's argument against the Boulder Project should not be taken as conclusive until more is known about alternative projects in the Lower Basin.

Why begin at Topock? The Mohave Valley is the hottest place known, at least so far as the U. S. Weather Bureau reports show, and the evaporation rate must be excessive; the potential power at that site is unequal to the immediate demand, and the foundations have not been drilled. It is likely that bed-rock will be found considerably deeper than at the Topock railroad bridge, where the maximum depth is not known, except that it is in excess of 100 ft. The Bridge Engineer of the Santa Fé Railway System has stated that "the east pier supporting the main cantilever and anchor arms goes down very close to 100 ft. and is founded on a bed of cemented volcanic boulders." It is not reassuring to contemplate laying the foundations at a depth of more than 100 ft. and at the same time contending with the untamed river in a canyon only 240 ft. wide. The writer believes that no attempt should be made to construct this or the Boulder Dam until considerable river control is secured farther up stream at sites where the bed-rock is shallow. The Topock Reservoir, too, would accumulate silt rapidly, especially on the higher levels, and the storage capacity would be reduced seriously in 20 years unless the Boulder Dam is built in the meantime. Of course, it would be a great advantage to the Imperial Valley to have the river desilted, but, likewise, it would be advantageous to those who operate turbine water-wheels in the Canyon Section.

Mindful of the need of quick action in the interests of the Imperial Valley, it seems most untimely for the Engineering Profession to propose a new location for the first dam, and to ask for another year of delay, to permit of determining the foundations at this new site.

The first dam should be at the Dewey site on the Grand River, built by the Federal Government as a regulating dam for flood control. Plans for such a dam were detailed by the U. S. Reclamation Service several years ago. With a height of 215 ft. it would create a storage capacity of 2 300 000 acre-ft. The depth to bed-rock is only 44 ft. below the surface. The rock is hard, Carboniferous sandstone, suitable for building operations. The site is above the silt-gathering area and, consequently, the life of the reservoir will be long. Studies of the effect of this proposed reservoir on flood flows showed that in 1914, a year of exceptionally high June floods, the crests at the mouth of the Grand River would have been reduced from 64 000 to 23 000 sec-ft. The regulation of the river effected by the Dewey Dam would give greater security in the construction of other dams farther down stream, and would double the power capacity at Diamond Creek and at other power sites above the mouth of the Virgin River. The site is below all irrigable areas on the Grand River, so that its use for river regulation would not interfere with the irrigation development of the Upper Basin. Its cost is estimated at $11 000 000.

It can be completed in three years. After other dams have been built on the river, the Dewey Project can be used, in part, for power.

By increasing the height of the dam to 245 ft., the storage capacity would be increased to 3 000 000 acre-ft. This would necessitate moving about 8 miles of the Denver and Rio Grande Railroad. The topography, however, is favorable, and the cost would be not more than 5% of the cost of the dam.

The Dewey Reservoir alone does not provide the 4 000 000 acre-ft. of storage required for flood control. The additional storage can be provided on the Green River, either at Flaming Gorge or at Juniper Mountain and Flaming Gorge. It is almost certain that the Flaming Gorge Dam will be built by the Utah Power and Light Company. The Federal Government should provide for a 30-ft. increase in height of the dam to give 1 000 000 acre-ft. of storage for flood control only. It is within the province of the Federal Power Commission to require this if the funds are provided by Congress or by projects farther down stream that will be benefited.

The most pressing problem of the Imperial Valley is not flood control, but the increase in the late-summer water supply. The shortage was acute and caused much damage in 1915, 1919, and 1922. The flow at present (July, 1924) is lower than on the corresponding date on any one of those years, and forecasts a severe shortage this year. Another year with as little run-off as 1902 would cause a complete loss of summer crops. The Dewey Reservoir alone would provide an ample late-summer water supply for at least 20 years. It would be filled each year in May and June and emptied gradually during the next six months.

It has been claimed that the first reservoir should be on the lower river below the mouth of the Virgin, so as to regulate the floods from the Virgin, the Little Colorado, and the San Juan. High floods on those streams are short-lived and are flattened out before they reach Yuma. The highest flood ever recorded on the San Juan occurred in 1911 and was estimated at 150 000 sec-ft. The maximum flow at Yuma during the period was only 60 000 sec-ft., of which only a part was due to the San Juan flood. The Little Colorado flood of September, 1923, was exceptionally high, but the discharge at Yuma did not exceed 60 000 sec-ft. Of course, those floods might have occurred coincidently, but it should be borne in mind that for a long time to come the levees of the lower river will be maintained for protection against floods of 200 000 sec-ft. from the Gila River. Furthermore, within recent years, an excellent system of flood warnings has been perfected, and track and quarry facilities at Andrade have been made very effective for fighting floods.

The cost of the Dewey Dam, or a part of the cost, should be pro-rated by the Federal Power Commission among the down-stream projects which will be benefited. The Diamond Creek Project could well afford to pay the entire cost, but the Boulder and other projects should each pay a share, because each will be benefited by it. The Federal Government should contribute to the cost of this and other dams on the head-waters of the Colorado under the same theory of government as was exemplified in the building of thirty-three dams on the Ohio River, that is, to secure river regulation and control, to make the stream

manageable and utilizable; or as exemplified in the purchase of great areas of land on the head-waters of the Allegheny and Monongahela for reforestation. Navigation is no more vital to the economic and social welfare of the group of six States bordering the Ohio than is the taming and harnessing of the Colorado to the welfare of the seven States along its course. The Federal Government should pay one-half the cost of the Dewey Dam and one-half the cost of securing 1 000 000 acre-ft. of storage for flood protection at the Flaming Gorge site.

It is desirable to have a larger measure of flood control, especially for such years as 1884 and 1909, and it is to be expected that, in time, the stream flow of the Upper Basin will be fully equated, probably at the mouth of the Green River or in the general vicinity of Lees Ferry. It is a disappointment to the writer that the author has not presented a definite plan for complete regulation of stream flow in the Upper Basin, in consonance with the general principles laid down by him. Until a general plan for complete regulation of the flow in the Upper Basin has been adopted, it would be a grave misfortune to give up the Ouray Reservoir site by restoring it to entry and granting a right of way for a railroad to cross it. Test borings that go all the way to bed-rock should be made at the Junction site, and borings should be made at Bluff.

Just now the urgent problem is to hold back somewhere a part of the June flood, and this can be done by means of reservoirs in the Upper Basin or anywhere above Parker. From the standpoint of protection to the Imperial Valley, the location of the storage is not so important as its immediate availability. The Imperial Valley should not be obliged to wait eight, ten, or twelve years for relief.

The development of the Colorado, as here proposed, by a moderate amount of storage, which can be utilized in Utah, Nevada, Arizona, and California, will prevent the early application of water to great areas in Sonora and Lower California and the subsequent claims of priority rights for those lands. It will retard development in Mexico until projects in Arizona and in the Upper Basin can be gotten under way.

The main arguments used by the proponents of the Boulder Project against storage in the Upper Basin are: (1) That it would leave the run-off from a large part of the water-shed unregulated; and (2) that the Upper Basin can make full use of the reservoirs situated above the canyon region. These arguments are not sound as has been shown herein. It is true that in order to prevent waste of water to the sea, there must be some reservoir farther down stream, but it will be many years before close control is required.

The First Power Project.—The Diamond Creek Project, at the mouth of Diamond Creek, 16 miles north of Peach Springs, on the Santa Fé Railway, is strictly a power project. If the Dewey Dam is built at once, the Diamond Creek Project might well be the first power development for the Lower Basin States. This project has a long list of advantages. It has a shallow depth to bed-rock, indeed, the bed-rock is swept bare during high flood stages; it is the narrowest between canyon walls; the walls are of granite;

it is easily accessible from the railroad and has convenient space close by for construction operations; and its distance from the center of gravity of the power market is little more than the distance from Boulder Canyon. The power installation of the Diamond Creek Project with the unregulated river flow will be between 200 000 and 300 000 h.p., depending on the height of the dam. If the Dewey Dam is built, an equal amount of power will be added, and the power possibilities will be further increased by the Flaming Gorge Project. A large part of this power would be marketed in California.

Arizona needs the Diamond Creek Project, but there is no urgency analogous to the situation in the Imperial Valley. Arizona has excellent power projects on the Salt and Verde Rivers and one at Yuma; the development of these projects is under way or is planned. The San Carlos Project of the U. S. Indian Service on the Gila River should include power development. Besides, it is a political question as to who should build the projects on the Colorado, whether private or public agencies, and, in this struggle, each side seems to be very successful in preventing progress by the other.

Assuming that storage for stream regulation has been obtained above the Canyon Section, then the problem in the Lower Basin involves the following desiderata:

1.—Power development at an early date and at minimum cost; each project to be part of a comprehensive plan that will utilize the maximum feasible head.

2.—Silt storage; this is of doubtful importance, as the silt problem will be less acute when the Imperial Canal is connected to Laguna Dam.

3.—Re-regulation; although not essential now, it will become so as the development of irrigation increases.

4.—Minimum loss by evaporation from reservoirs; this also will become important as irrigation development advances.

These conditions are met at the Boulder site far better than at Topock; and, as a first power project, the Diamond Creek location is better than the one at Boulder, assuming, of course, that regulation of the river is effected in the Upper Basin. The importance of making power available at an early date cannot be over-estimated. Despite the generous surplus of power for 1924 shown in Table 14, there is a critical power shortage throughout Southern California now (July, 1924), and it is doubtful whether the supply will again equal the demand until power is obtained from the Colorado. The growth in population, or in use of power, is not the only factor; the amount of rainfall, the price of fuel oil, and other elements enter, many of which cannot be forecast at all. It is unlikely that power from the Boulder Project can be obtained in less than 10 years; power from Diamond Creek can be had in 4 years, if the deadlock between those interests that are seeking to control the development can be broken.

The author has referred repeatedly to the need of conforming to one optimum comprehensive scheme of development. There can be no objection on this ground to the immediate development of the Dewey and Flaming Gorge Projects and probably none to the Diamond Creek Project.

Louis C. Hill,* M. Am. Soc. C. E.—It is generally agreed that the Colorado River must be controlled not only for flood protection but for irrigation and the development of power. As stated by the author, however, protection of the lands along the river and in the Imperial Valley from damage by inundation is of primary importance. No plan that limits this protection should be considered.

The determination of what ultimate development of the Colorado River will give sufficient protection from floods, will irrigate most economically the maximum area, and will return the largest net income from the generation of power, is a problem not yet solved. Any schedule of development leading to such ultimate control and use of the waters of the Colorado River will be greatly influenced by such economic factors as the rate of agricultural development within the Basin and the rate of increase in the use of power throughout the entire Southwest.

As the generation of power will be of secondary importance to irrigation, and as both must be considered in the light of their effect on the control of floods, the speaker will touch on these subjects in that order.

Power Development.—If the entire Southwest were thickly settled and industrially developed it would be logical to create the first storage reservoir on the Colorado near the upper end of the Canyon Section. The regulation of the flow from this reservoir would greatly increase the available power at each site below. Furthermore, this regulation would reduce the cost of foundation work on any dams built subsequently in the canyon.

Before being called by the U. S. Bureau of Reclamation and by private power interests to study the control of the Colorado River, the speaker was of the opinion that the Glen Canyon site should be developed first, in spite of the distance to a power market. However, after personal investigation of this site at the head of the canyon and of the Boulder Canyon or Black Canyon sites at the lower end, he is convinced that, under the physical and economic conditions, the net advantages of the lower site outweigh those of Glen Canyon as the location of the first large reservoir. Glen Canyon is about 100 miles farther from a railroad than Black Canyon and 250 miles farther from the principal power market.

Although the depth to bed-rock at Glen Canyon is about 80 ft. as against 124 ft. at Black Canyon, there is considerable doubt as to the suitability of the Glen Canyon sandstone for the foundation of a dam as high as that required for proper control of the Colorado River. It is no doubt true that tests of dry samples indicate ample compressive strength, but this material will be subjected to high hydrostatic pressure so that its action when wet must be considered. Samples of this sandstone were secured by the speaker himself from an excavation made in the side wall of Glen Canyon by the Southern California Edison Company for the purpose of obtaining fair samples. After one of these rock fragments has been in water for a few minutes, it may be easily broken and crushed into sand particles with the fingers.

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Even if careful and extended tests should show sufficient strength and resistance to percolation to warrant the erection of a high dam, this sandstone is unsuitable for use in the construction of either a rock-filled or a masonry dam. The aggregate would have to be obtained several miles down stream from the dam site.

In his conclusions, the author has stated that a high dam should not be built in Boulder Canyon or Black Canyon because the ultimate power development under the U. S. Bureau of Reclamation plan would be curtailed by 300 000 h.p., due to the loss of 80 ft. of head between the Boulder Reservoir and Diamond Creek. The selection of Black Canyon as the site of a storage dam should not be condemned arbitrarily because the Federal Power Commission believes that too high a dam is provided in the preliminary plans of the U. S. Bureau of Reclamation.

In all the plans proposed by the author, he has considered that the elevation of the head-water above each dam should equal the tail-water level of the next dam. The least lowering of any reservoir must result accordingly in the loss of just that amount of available head in the development of power.

The requirements for irrigation in the Lower Basin will seldom correspond to the quantity of water that will be released from the upper storage reservoirs for the development of power. Consequently, re-regulation of the water released for power must be effected in some reservoir at the lower end of the Canyon Section. As this re-regulation will result in unavoidable fluctuations in the level of the lowest storage reservoir, provision must be made in the ultimate development to prevent a corresponding loss of head and power.

It has been suggested by Mr. R. A. Hill that a dam, which would be sufficiently high to back water to the foot of the power dam at Diamond Creek, should be constructed ultimately across the upper end of Boulder Reservoir, probably at either Spencer Canyon or Bridge Canyon. The normal low-water level in Boulder Reservoir should correspond to the river level at this intermediate dam, and during periods of high water this dam should be partly submerged. In spite of inevitable fluctuations in the level of Boulder Reservoir, it would be practicable to use in the development of power all the head between these reservoirs by the installation of variable head turbines arranged to operate when submerged as much as 100 ft. Furthermore, this plan would eliminate the loss of 80 ft. of head below Diamond Creek, cited by the author as one of the main objections to a high dam at Boulder Canyon or Black Canyon.

It will be many years before the construction of this intermediate dam becomes necessary, but the speaker believes that it should be considered as part of the ultimate development on the Colorado River. It would not be difficult to construct; the flow of the river from above would be under control; and Boulder Reservoir would periodically be low enough to unwater the dam sites in Spencer and Bridge Canyons.

In order that the maximum beneficial use may be made of the waters of the Colorado River, a large amount of storage to equalize the flow over several years must be provided for irrigation as well as for power. Although the

author has given some weight to this in his plan for the development of power, he has not provided sufficiently for irrigation, which should be given prior consideration.

Irrigation.—Water storage for irrigation should be as close as possible to the lands to be served. If this were the only consideration, the selection of Boulder Reservoir—to be created by a dam in Black Canyon—would be without material opposition as this reservoir is only 300 miles above the Laguna Diversion Weir, as compared with nearly 700 miles for the Glen Canyon site.

Storage reservoirs above the Canyon Section should be built as the need arises, and should be operated to best satisfy the power demand. These reservoirs, when the ultimate development has been effected, will practically equalize the seasonal variations in the flow of the Colorado and will reduce the annual variations. Elimination of annual variations is of primary importance in the conservation of water for irrigation; consequently, there must be provided below the Canyon Section as much irrigation storage as is economical.

There is no clause in the Colorado River Compact which provides for the release in a year of low flow of water for the irrigation of lands below the Canyon Section, although it does provide that during 10 consecutive years a total of 75 000 000 acre-ft. must be allowed to reach Lees Ferry. Consequently, a large amount of hold-over storage must be available to assure the lands in the Lower Basin of a continuous supply of water for irrigation.

The speaker is of the opinion that a reservoir of sufficient capacity for these purposes can be created most economically at Boulder Canyon. For many years it will be possible to release water from Boulder Reservoir in such quantities and at such times as will satisfy both the power and irrigation demands. Ultimately, the release of water from Boulder Reservoir will approach the variable irrigation requirement of the Lower Basin, except as modified by the regulatory effect of a reservoir farther down stream. The power output of other plants along the Colorado would be so modified as to satisfy that part of the power demand not taken care of by the output at Boulder.

Secondary re-regulation of the normal release from Boulder Reservoir should be provided as close as possible to the lands to be served with water for irrigation. As a comparatively small capacity will be required, the best location, as far as is known at present, would be just below the mouth of Bill Williams River. A dam at this point would also divert water into canals to irrigate large bodies of land between Parker and Yuma, Ariz., and in the Palo Verde Valley. This reservoir would also serve as a settling basin and point of diversion for an additional domestic supply for the cities and towns of Southern California the local water resources of which have been exploited to the safe limit. The value of this reservoir for flood control will be discussed later.

The author has considered that re-regulation can best be effected with a dam at Topock, creating the Mohave Reservoir. The only objection to this

site, in his opinion, is that its flowage damages may be high, as it will require moving the Town of Needles, a division point on the Santa Fé Railroad, and re-locating 15 to 20 miles of main-line railroad. He neglected to call attention to the fact that about 100 000 acres of land in the Mohave and Cottonwood Valleys would be submerged and thus eliminated from agricultural development.

The principal advantage claimed for the Mohave Reservoir is based on rather fallacious reasoning. The author states that water for 50 000 acres of land would be saved if Boulder Reservoir were reduced in size and Mohave Reservoir constructed. This assertion is based on apparent differences in evaporation losses.

According to Table 18, there would be, under the U. S. Bureau of Reclamation plan, an evaporation loss at Boulder Canyon of 650 000 acre-ft. per annum, or 600 000 acre-ft. more than at present, while future evaporation loss, under the author's plan, with the same flood control at Mohave, would be 490 000 acre-ft., from which has been deducted 350 000 acre-ft. as the present loss, leaving only 140 000 acre-ft. as the net loss.

It is incorrect to credit all the present loss from the 72 000 acres of flooded area in Mohave and Cottonwood Valleys to the Mohave Reservoir. Flood control in Boulder Reservoir would greatly reduce the area from which evaporation would occur. This inevitable reduction should properly be credited to the Boulder Reservoir and only the remainder should be a credit to the Mohave Reservoir. Unless submerged, these valleys will be irrigated when regulation is provided, consequently the evaporation from this irrigable area cannot justifiably be deducted from the evaporation loss from the reservoir which would submerge these lands. It would be equally logical to create a reservoir out of the cultivated Palo Verde and Parker Valleys in order to eliminate the evaporation loss from these lands.

In Table 33 is shown the manner in which the speaker believes these losses should be considered. For purposes of comparison, the values in Table 18 are taken as correct.

This saving by the elimination of Mohave Reservoir would be about sufficient to compensate for the evaporation loss from the proposed secondary reservoir near Parker. At least the quantity of irrigation storage given must be provided below the Canyon Section, in addition to that needed for flood control and for silt accumulation, even if up-river storage is developed later, as considered by the author. Careful analysis of all the factors affecting the reduction of the water supply by evaporation shows clearly the fallacy of this objection to Boulder Reservoir.

Flood Control.—The ideal reservoir for flood control would be below the last tributary of any consequence. Unfortunately, such a provision on the Colorado River is impossible below the Gila, so that the control of the Gila River floods must be by separate regulating works on that stream. It is generally agreed that reservoirs for flood control on the Colorado should be at, or below, the lower end of the Canyon Section. Consequently, discussion revolves around the selection of Boulder Reservoir or Mohave Reservoir.

There is practically no contributing drainage in the 80 miles along the Colorado River between the Black Canyon Dam site and Topock, hence this factor is negligible in the selection of the proper site.

TABLE 33.—Comparison of Evaporation Losses.

	Acre-feet, per annum.	
Boulder Reservoir with Maximum Level at Elevation 1 250, Providing 34 000 000 Acre-ft. of Storage, of Which 8 000 000 Acre-ft. are for Flood Control:		
Loss from reservoir surface	650 000	.....
Present loss from stream bed	.....	50 000
Reduction in flooded area above Topock assumed at only 50% and corresponding reduction in evaporation loss, one-half of 350 000 acre-ft.	.....	175 000
Total	650 000	225 000
Additional evaporation loss under Reclamation plan	.....	425 000
Mohave Reservoir with Maximum Level at Elevation 605, Providing 13 000 000 Acre-ft. of Storage, of Which 8 000 000 Acre-ft. are for Flood Control:		
Loss from reservoir surface	490 000	.....
Loss from Boulder Reservoir with storage capacity of 10 000 000 acre-ft. according to author's Plan No. 3	340 000	.....
Allowance of 50% of present loss from flooded area within proposed Mohave Reservoir	.....	175 000
Present loss from stream bed at Boulder Reservoir (Plan No. 3)	.....	40 000
Total	830 000	215 000
Additional evaporation loss under author's plan	.....	615 000
Net evaporation loss chargeable to Mohave Reservoir	615 000	
Net evaporation loss chargeable to Boulder Reservoir	425 000	
Difference in favor of Boulder Reservoir	190 000	

Floods from the drainage area of the Gila River have as great a maximum discharge as floods in the Colorado above their junction at Yuma, but the Gila floods are of short duration. Under present conditions these floods do not tend to coincide, as the Colorado is at its highest stages from May to July, while the Gila floods almost invariably occur during the winter. The former result from melting snow on the mountains in Utah, Colorado, and Wyoming; the latter are from heavy rains in Arizona and New Mexico.

When the Colorado floods are reduced in magnitude by regulation, the flow of the river in the late fall and winter will be increased materially. Provision must be made, therefore, to minimize the probability of a Gila flood being augmented by 40 000 sec-ft. or more from the Colorado.

In this connection, Mohave Reservoir would have an advantage as it is about one day closer to Yuma than Boulder Reservoir. This advantage would be entirely offset if a dam creating the proposed Parker Reservoir were constructed below the mouth of Bill Williams River. Not only would this regulating reservoir be another day closer to Yuma, but it would directly control the flashy floods from the Bill Williams.

When storms were occurring on the head-waters of the Gila River, the normal release from Boulder Reservoir would be curtailed. When it became

evident that a flood was imminent on the Gila, the entire flow of the Bill Williams, and that water already released from Boulder Reservoir, could be held back in the Parker Reservoir until the Gila flood had receded. As Parker is closer to Yuma than the last important tributary on the Gila, such a method would be entirely practicable.

The author has resorted to another method of handling the floods of the Colorado River, which does not meet with the approval of engineers long experienced in the actual control of these floods. In order to avoid the possibility of a dangerous coincidence in the flows of the Colorado and of the Gila at Yuma, Colonel Kelly proposes that the stored flood waters shall be released at the rate of about 80 000 sec-ft. By so doing, all flood waters would have been removed from the reservoir before the season for Gila floods. The hydrograph, Plate I, shows that this would be entirely unnecessary in more than half the years.

As intelligent operation of the flood-control reservoir would practically obviate the coincidence of floods from the Gila and the Colorado, the speaker can see no sound reason for disregarding the opinion of those who have personally been combatting the floods of the Colorado. The engineer of the Palo Verde District has stated—according to the author—that if no levees are provided, the lands of this district would be flooded when the flow exceeds 50 000 sec-ft. and that bank protection is required at floods of more than 35 000 sec-ft. It is the opinion of the engineers of the Yuma Project, that floods of the Colorado should be reduced to 40 000 sec-ft. and that a continuous flow of 80 000 sec-ft. would be almost as hazardous as no control whatsoever. The average flow of the Colorado after regulation is effected will be from 15 000 to 30 000 sec-ft. No channel maintained by such a flow will carry a flood discharge of 80 000 sec-ft. unless both levees and bank protection are provided.

Conclusions.—It is the firm opinion of the speaker that when all factors are taken into consideration, it will be found that the Boulder Canyon Project will most nearly satisfy the physical and hydraulic conditions and at the same time will fit in best with existent economic conditions. The U. S. Bureau of Reclamation plan should not be adopted without modification, neither should irrigation be subordinated for power, as would be the case under the plan of development advanced by the author.

The speaker has made no reference to the All-American Canal, as he considers that this matter particularly concerns the Imperial Valley and that it should be entirely divorced from the general development of the Colorado River. From the attitude of the Mexican Commissioner at the time the speaker conferred with him as American Commissioner on the allotment of the waters of the Colorado River and of the Rio Grande, it is quite probable that an equitable settlement can be made which will be satisfactory to both Mexico and the United States. Such a treaty should be negotiated as soon as possible.

All efforts should be concentrated toward the immediate construction of a dam in either Boulder Canyon or Black Canyon that will fit into the ultimate plan of development, while creating for immediate use a reservoir of

sufficient capacity for the accumulation of silt, for flood protection, for ample irrigation and domestic supply, and for the development of enough power to satisfy the demand for many years. The selection of the agency or interests which shall effect this construction is a matter entirely foreign to the determination of the proper plan of development of the waters of the Colorado River.

C. S. Jarvis,* M. Am. Soc. C. E. (by letter).—This paper and the discussions that have followed it are so timely and have done so much toward focusing attention upon the outstanding issues, that the main problems are several steps nearer solution.

One of the important points brought out by the author in regard to the feasible irrigation development is that the ultimate depletion of the water supply due to the use in the Upper Basin will probably never exceed 5 000 000 acre-ft., or approximately 1 acre-ft. per acre supplied. Relating this fact to the apportionment of 7 000 000 acre-ft. to the Upper Basin under the terms of the Colorado River Compact as now drawn, it is apparent that although they may divert the prescribed quantity, the return flow should account for nearly 30%, or 2 000 000 acre-ft., to benefit the lower projects. Also, the return flow from the projects in the Lower Basin and above Hanlon Heading will inure to the benefit of the Imperial Valley and lands in Mexico.

There seems to be no possible challenge to the author's statement of principles regarding the desirable locations for storage, and he plainly concedes that compromises have to be made to suit the natural conditions, such as available sites of sufficient capacity, dependable supply, and prior developments and rights. Inasmuch as the triple interests of irrigation, power projects, and flood protection must be served, it is evident that both upper and lower storage sites must be utilized. Most of these sites near the head-waters have such limited capacity as to be well within the scope of private enterprise.

The investigations of the U. S. Reclamation Service during the past two decades furnish the most definite data yet made available as to comparative values of the various plans suggested. However, they were not extensive enough to determine the actual merits of the principal projects now advocated.

It appears advisable to include an extensive examination of the proposed reservoir basins, especially that which would be utilized by the Boulder Canyon Project, because of the known deposits of rock salt and other soluble minerals which might prove harmful to the lands in the lower deltas. Two of the known salt ridges lie somewhat above the proposed high-water level of the plan advocated by the Reclamation Service; but the extent of the saline deposits within the flooded area should be known in advance.

If some large storage reservoir is provided, such as has been suggested at Glen Canyon, Boulder Canyon, or the Mohave site, then, in order to realize on the power feature during the non-irrigating season, water must be wasted. This would make advisable the provision for auxiliary storage at some point down stream. Thus, the Bulls Head or the Parker site would be among those

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examined for the purpose of retaining a relatively small storage, from which no primary power would be obtainable.

The conflicting views as to whether the supply of water is greater or less than required by the available irrigable lands, are evidently traceable to varying judgments concerning lands suitable for reclamation.

Field observations made by the writer on the head-waters and in the intermediate valleys of various tributaries justify the opinion that all the feasible sites at the higher levels should be utilized as rapidly as they can be financed. This is particularly true of the Gila River in Arizona, where the direct benefits to irrigation and the incidental reduction of silt burdens and flood menace for the lower delta will be of immense value.

About twenty years ago field data were collected and designs made for the Gila under the direction of Arthur P. Davis, Past-President, Am. Soc. C. E., and others. The use of this material should prevent the delays and controversies over the Colorado River problem which at present seem to be impending.

E. W. Lane,* Assoc. M. Am. Soc. C. E. (by letter).—The effect of the deposition of silt on the storage capacity of the proposed basins in the Colorado River has been given careful consideration, but has any study been made of the effect of discharging into the river below the flood-control dam of from 40 000 to 80 000 cu. ft. per sec. of water from which the silt has been removed? It is obvious that the water will pick up and carry away large quantities of silt, and although there is probably little, if any, precedent for determining the quantity thus removed, it is easy to show that this may be important.

The silt now passing the Boulder Dam site is estimated to be 80 000 acre-ft. per year. Neglecting the effect of the redistribution of flow and diversions, if it is assumed that before traveling the 459 miles to the Gulf of California, the stream has picked up as much silt as it is capable of carrying, it would remove practically 80 000 acre-ft. of silt per year. Assuming a river bed 1 500 ft. wide, this quantity would be equivalent to an average depth of nearly 1 ft. per year. It would seem more reasonable to expect it to pick up this in the first 100 miles, in which case, the average lowering would be about 4.5 ft. per year; at the upper end it might be twice this, or more. As the silt percentage curve (Fig. 10) given by the author shows that the silt content was nearly the maximum at a discharge of 40 000 cu. ft. per sec., the redistribution of flow might even increase the silt removal more than 80 000 acre-ft. per year.

The quantity of silt removed from the river bed below the dam will be influenced by a number of factors. As the river bed is cut down, the tributary streams will cut down their beds also, bringing into the main stream large quantities of silt which will reduce the rate of lowering of the main stream bed. The fewer and smaller the tributary streams, therefore, the more rapid will be the lowering of the main stream bed.

If bed-rock lies close to the river bottom or heavy gravel deposits exist, such conditions will retard or limit the erosion in their vicinity, but this will only tend to increase it in other parts of the stream. Diversions of water, of

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course, will reduce the lowering, by removing the eroding and transporting agent.

The effect of lowering the bed of the Colorado River, down stream from the storage dam, would be in part harmful and in part beneficial. It would lower the tail-water of the power house at the dam; this would give an increase of head, but it might destroy the effectiveness of the draft-tubes, if they were not designed for such a contingency. This effect may also occur at the proposed power dams on the Colorado.

If the lowering is considerable, it will make the diversion of water for irrigation more difficult and may cause serious undermining of the existing dams founded on silt.

Although it is not possible to predict how much effect this lowering will have in improving flood conditions in the lower river, it will at least be beneficial, and may become, in the course of a number of years, an important factor. It is possible that if engineers knew more of the laws governing this action, by preventing the cutting back of the tributaries, and possibly building a retarding basin on the Gila River, to remove the silt from its water also, the river might be made to excavate its own channel deep enough to carry the flood flow without endangering the surrounding country. In his discussion* of the paper entitled "Flood Problems in China", by John R. Freeman, Past-President, Am. Soc. C. E., the writer has already suggested that this method is worthy of study for the improvement of the Yellow River in China, where the conditions are particularly favorable.

Raymond A. Hill,† Assoc. M. Am. Soc. C. E. (by letter).—The problem of the Colorado River is too complex for arbitrary solution at this time, as the data which are available, or which might be made available, cannot include certain important factors the determination of which is necessary to the proper solution. The extent of flood control which will be needed after storage is provided, and the quantity of water which will be required for irrigation, are two such indeterminates.

The author has made use of certain curves showing the relation between the percentage of silt and the discharge of the Colorado River at Yuma, the relation between velocity and discharge, and the area-discharge relation. These curves were originally prepared by the writer in connection with certain studies to determine the relation between silt content and depth for various velocities. Fig. 24 shows the volume of silt which is transported by the Colorado River at various stages when there is no contributing flow from the Gila River.

It would seem from this that the author is justified theoretically in maintaining that the Colorado River has been more stable at 80 000 sec-ft. than at 40 000 sec-ft. It must be realized, however, that during the years in which these silt measurements were made the river was constantly fluctuating in volume, particularly at the higher stages, and that even with a silt-laden river the proportion of silt would tend to decrease if the discharge were held constant.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1558.

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This in itself would alter the stage of equilibrium. Certainly the characteristics of a desilted river cannot be based on data acquired when the river was heavily laden with silt. Before the Colorado River is regulated, no one can predict which rate of discharge from a flood-control reservoir will produce the most stable condition.

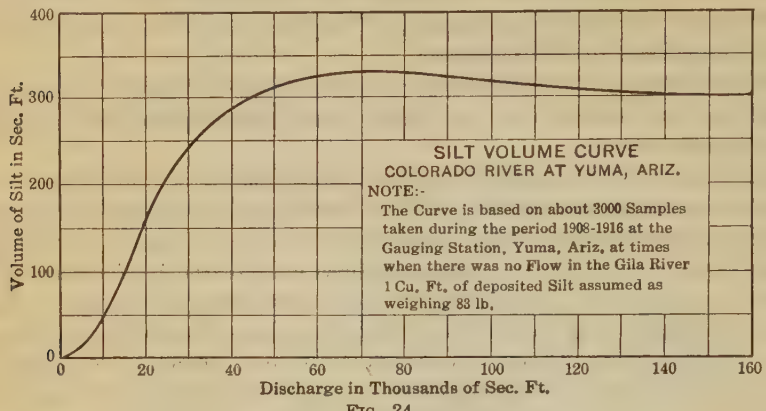

FIG. 24.

Silt which enters the Colorado River above the last reservoir will be intercepted and deposited in this and other reservoirs. The water which is passed down the river will be clear at first, and it will be highly desirable to keep it as clear as possible by reducing scour in the channel. A load of silt, however, will be picked up along the river whenever the rate of discharge is materially increased. As much of this silt will be carried into the irrigation canals, fluctuations in the flow of the Colorado River should be reduced to a minimum. This consideration alone will require that the release from the flood-control reservoirs be held to much less than 80 000 sec-ft., as the irrigation demand will not exceed about 30 000 sec-ft.

Estimates of the available water supply and of the future irrigation requirements naturally differ widely, as these are to a considerable extent matters of conjecture.

The average annual discharge of the Colorado River at Yuma, as estimated by Herman Stabler, M. Am. Soc. C. E., was 30% greater for the last 20 years than the average for the preceding 20 years. It is believed that the run-off for the years preceding 1902 has been computed too conservatively.

Prior to its break into the Imperial Valley, the Colorado River had been lengthening its old channel by deposition of silt at the head of the Gulf of California. This naturally resulted in a continuous raising of the water surface at Yuma, until, as shown in Fig. 4, only about 30 000 sec-ft. were carried at a gauge height of 23.0 ft. Since 1905, largely on account of the change in the river channel below Yuma, the average discharge for the same gauge height has been more than 50 000 sec-ft. The original gauge readings were kept by the Southern Pacific Railroad Company which had little interest in the volume of water in the Colorado River. Consequently, there were not

enough actual stream-flow measurements, prior to 1902, to determine the continually changing gauge-discharge relation. As the river bed was steadily rising, calculations based on gauge heights would tend to give results lower than the actual discharge of the river.

Fortunately, records covering many years of measured run-off will be available before the irrigable acreage will have expanded sufficiently to absorb the increase in supply which will come with regulation.

In Table 11 the author shows a compilation and classification of the areas which may be served from the Colorado River. His conclusion that the water supply is obviously inadequate depends on the accuracy of the calculations as to the water supply, on the economic feasibility of agriculture on these lands, and on that use of water assumed as necessary.

So much of the area included is less suited to agriculture than other large undeveloped sections in the United States and Mexico, that the writer, who is familiar with the entire area, cannot conceive of its complete development. In 1907, 190 000 acres were irrigated on all the projects of the U. S. Bureau of Reclamation; by 1921, the irrigated area had increased to only 1 250 000 acres, in spite of the fact that natural and economic conditions were favorable to rapid expansion. It has taken 20 years for less than 500 000 acres to be placed in cultivation in the Imperial Valley of California. When it is realized that these areas were the easiest and cheapest to develop, it should be obvious that a great many years must elapse before the irrigated area below the canyon will exceed 2 000 000 acres.

Conservation of water by the reduction of the gross duty will more than offset any further increases in the irrigated area. The author has assumed that 4.2 to 4.5 acre-ft. of water will be consumed each year on every irrigable acre below the canyon. The actual consumption per gross acre of irrigable area in that part of the Imperial Valley served by the Imperial Irrigation District is shown in Table 34, summarized from the 1923 report of the Chief Engineer of that District.

TABLE 34.—Water Delivery in Imperial Valley, 1923 Season.

Division.	Irrigable area, in acres.	Water delivered to farms, in acre-feet.	Acre-feet per gross acre.
Calexico	47 000	99 887	2.12
Holtville	85 000	154 683	1.82
Imperial	130 000	249 630	1.92
Brawley, East	78 000	156 188	2.00
Brawley, West	82 000	149 933	1.83
Calipatria	93 000	109 909	1.18
Total	515 000	920 230	1.79

As about 80% of the irrigable area was actually cultivated in the Calexico Division, 70% in the Holtville, 70% in Imperial, 75% in Brawley, East, 70% in Brawley, West, and 45% in the Calipatria Division, the net duty for each acre actually cropped was approximately 2.7 acre-ft. The gross duty for each

irrigable acre of a fully developed area will be almost equal to the net duty on each cropped acre, as the transmission losses are generally counterbalanced by the difference between the irrigable and cropped areas.

Even without further improvement in irrigation and farm methods, at least 1.5 acre-ft. per acre, or about 5 000 000 acre-ft., should be deducted from the estimated total requirement. Projects comprising several million acres in the San Joaquin Valley of California are being carried forward on the basis of 2.0 acre-ft. per acre, and this quantity is considered sufficient in Southern California. As the character of development progresses, the economic advantages arising from the minimum use of water will force a similar procedure in the Colorado River Basin.

Therefore, it is the writer's firm opinion that future experience will disprove the contention that the water supply from the Colorado River will be insufficient for much of the lands, because he believes that the water supply has been under-estimated, that the economically feasible area has been overestimated, and, finally, that the available supply can serve a far greater area than that claimed by the author.

E. B. Debler, Esq.* (by letter).—According to the paper, little, if any, saving in the cost of bank protection and maintenance will result from reducing the flood discharge of the Colorado to less than 80 000 sec-ft. Porter J. Preston, M. Am. Soc. C. E., Project Manager and Superintendent of the Yuma Project since 1920, and R. N. Priest, Assoc. M. Am. Soc. C. E., Construction Superintendent since 1903, most of the time in charge of river control work through the Yuma Project, advise that the levees begin to undercut at discharges of 50 000 sec-ft. and that overflow of the natural banks below Yuma begins at discharges of 30 000 to 50 000 sec-ft., depending on river conditions. Colonel Kelly states on the authority of the Engineer of the Palo Verde Project that flooding without levees would begin in the Palo Verde Valley with discharges of 50 000 sec-ft. and that bank protection is required for flows exceeding 35 000 sec-ft. These conditions are typical of all the valleys below the canyon region. Mr. Preston further states that the channel capacity is increased by 10 000 to 20 000 sec-ft. by scouring of the river bed with continuation of the flood.

Until the Gila River is controlled, levees will have to be maintained to the present levels, but the Gila floods are of such short duration that they do not require bank protection for the levees. The expenditures on levees have largely been for purposes of revetment and this is required by reason of the continued persistent undermining of levees by extended floods originating along the Colorado. In 1913, a levee was cut entirely through with a maximum flood discharge of 62 000 sec-ft. The advantages of reducing all ordinary floods to less than 50 000 sec-ft. are obvious.

Colonel Kelly recommends a flood-storage capacity of 4 000 000 acre-ft. to reduce recorded floods to 75 000 sec-ft. and to permit the interruption of flow for repairs in case of breaks in levees. His floods refer, however, only to the period since 1902. Reliable records indicate a flood in 1884 far in

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excess of any since 1902, with a maximum discharge for that year at Needles, estimated by Santa Fé railroad engineers on the ground, at 384 000 sec-ft. The maximum discharge at Laguna Dam since 1902 has been 186 000 sec-ft. Herman Stabler, M. Am. Soc. C. E., estimated the annual discharge for 1884 at 27 200 000 acre-ft. as compared with a discharge of 21 100 000 acre-ft. for 1920 on which Colonel Kelly's recommendation of 4 000 000 acre-ft. of storage is based. The difference of 6 100 000 acre-ft. between the discharges of these two years is largely water that would have to be stored if discharges at Yuma are to be maintained at 75 000 sec-ft. as recommended by Colonel Kelly. The required storage capacity to hold the 1884 flood to 75 000 sec-ft. would then be 10 000 000 acre-ft. compared with 4 000 000 acre-ft. recommended by Colonel Kelly and 8 000 000 acre-ft. proposed by the U. S. Bureau of Reclamation.

The problem of flood control is not one of water alone; it contains an element of possibly even greater importance—silt control. It may be granted that power dams, if constructed on the Colorado River in the immediate future, will cause Colorado River waters to leave the canyon region practically desilted. The present topography of the stream bed and valleys below Boulder Canyon is a balance between the silt supply brought by the stream from upper sources and existing channel conditions. When the silt supply is cut off, this balance is disturbed, and the result is entirely beyond the ability of man to forecast with certainty. Will it be safe to turn a continuous stream of 75 000 sec-ft. of comparatively silt-free water into a channel that has adapted itself to heavily silted water? To provide for expected, but wholly uncertain, developments from this change in character of stream flow, liberal allowance must be made for controlling the stream flow to discharges materially less than those proposed by Colonel Kelly.

The paper outlines a permissible development at Mohave Valley in case Congress should deem some provision for flood control and irrigation necessary, and places the cost of the power plant at $35 per h.p., wholly omitting the cost of transmission lines. Estimates for Boulder Canyon indicate a cost of $76 per h.p. for the power plant and pressure tunnels with a plant capacity of 200 000 h.p. With the lower head available at Mohave Valley, the cost at that point would be no less. Adding to this amount the estimated cost of $115 per h.p. for the dam gives a total cost of $191 per h.p. for a power plant nearly 300 miles from the Southern California market as compared with a similar cost for competitive California sites not only nearer the market, but located so that advantage may be taken of transmission lines already constructed and not fully loaded.

To produce this power the normal flow of the Colorado River must be increased very materially. If, for this discussion, it is admitted that the increase in normal flow caused by the construction of Boulder Canyon Reservoir would be a menace to the fullest possible use of Colorado River waters by reason of an immediate impetus in Mexican irrigation, the Mohave Valley plan for all practical purposes is open to the same objection.

The inability of power revenues at the Mohave Valley site to repay the construction cost is probably best indicated by the fact that this dam site is

the only one on the Lower Colorado River that has not been plastered with applications before the Federal Power Commission.

Colonel Kelly states that "all development needed on the Colorado will be built by private capital under adequate Federal and State regulation if the river is given over to development under the Federal Water Power Act". There is little reason to doubt that speedy development can be obtained in this way, along lines, however, as favorable as possible for power but with a minimum regard for irrigation and flood control.

The present status of the Flaming Gorge permit of the Utah Power and Light Company is an excellent example. This permit was originally granted, subject to up-stream irrigation use. It is now understood, however, that the Company is reluctant to accept license under these terms and requests irrigation development above the reservoir site to be limited.

The claim that construction of the Boulder Canyon Reservoir will jeopardize up-stream water rights by utilizing the total stream flow applies with equal force to any and all reservoirs and power plans proposed, as even the so-called up-stream reservoirs are below the irrigable areas in the Upper Basin.

Regardless of the amount of storage provided, the flow available for power, whether at Mohave or Diamond Creek, is made up of the normal flow plus a storage increment, the latter alone being affected by the storage capacity provided. The normal flow is of vital interest to power and to irrigation development throughout the basin. Up-stream irrigation development will reduce this plan to the detriment of power development below; it can be safeguarded only by legal and binding agreements which will preclude opposition to such depletion.

In case a plan of private development is adopted, a large development of Colorado River power may come about very quickly in comparison with irrigation development; the provisions for future irrigation development, therefore, must permit wide latitude from any plans that may now be outlined. If inadequate storage capacity is provided for flood control and irrigation before power developments become crystallized, the bar to full irrigation development becomes insurmountable. The proposal by Colonel Kelly and others to inundate a large irrigable area like the Mohave Valley as the first step in irrigation development requires more justification than has yet been advanced. On the other hand, an excess of storage for irrigation and flood control can be converted later to use for power development, resulting only temporarily in a reduction in power output, provided a market then exists for this added power. Although the Reclamation plan is severely criticized, no alternative plan is outlined in sufficient detail to indicate its comparative value or feasibility.

The author enunciates the generally recognized axiom that power development should be based on up-stream storage. Practical plans usually require some deviation from this principle either to obtain the greatest power output or to obtain a near maximum output at reasonable cost. It is in conformity with this practical application of the principle that the plans outlined in the discussion by Mr. Weymouth have been evolved.

It is granted that widespread up-stream storage would materially alter these plans, but thus far no agency has proposed such construction and the prospects thereof within a reasonable time limit are decidedly unfavorable. To hold back feasible developments on the Lower Colorado for fear they may not fit in with some theoretical ultimate plan is likely to cause far more loss from delayed development than the possible cost in the far future of reconstruction to fit in with changed conditions. Had the same principle been adopted in the construction of American railroads there would not be a single line from coast to coast to-day.

William Kelly,* M. Am. Soc. C. E. (by letter).—The writer has been much interested in the discussions by Messrs. Allison, Fowler, La Rue, Davis, Grunsky, Weymouth, Smith, Jarvis, L. C. Hill, R. A. Hill, Lane, and Debler, each of whom has contributed valuable data or opinions on one or more phases of the problem.

The writer will give a brief review of these discussions, and close with an expression of his conclusions.

Mr. Allison supports the views expressed in the paper with respect to flood control which, in view of his long connection with Imperial Valley, is very gratifying. He expresses the opinion that there will always be sufficient water for all future irrigation needs and presents data to substantiate this view. Many engineers disagree with him in this respect, but practically all now agree with his further view that power development will precede irrigation development and will materially aid the latter. Mr. Allison's presentation of data on and his discussion of the All-American Canal are valuable contributions, and should prevent the Federal Government from ever taking part in that project.

Mr. Fowler brings out clearly and accurately certain important functions of the Federal Water Power Act and their relation to the Colorado River, which explain the point of view from which the writer has approached the problem. Mr. Fowler's reasoning on the distribution of costs among the various interests instead of inflicting it all on the power consumer, is sound. Finally, he brings out more forcefully than the writer the advantages of a well-balanced progressive development over the single large Boulder Canyon plan.

Mr. La Rue presents data to show that all the resources of the Colorado will be needed and that probably there will not be enough water to irrigate all the available land. Mr. Allison and certain engineers of the Reclamation Service differ with him in the latter view, whereas Arizona and many engineers who have studied the question agree with him. Nobody can be certain what the future irrigation development will be. It may well be slow as predicted by Mr. Allison, but, considering the vast arid territory through which the river runs, the only safe policy is to see that any project built at present shall involve no ultimate waste of water such as will result from duplicate regulatory storage. Mr. La Rue's recommendations at the end of his discussion have great merit, and are worthy of serious consideration.

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Mr. Davis' discussion is a clever composition of facts combined with assertions and partial quotations that misinterpret the writer's views. His arguments appear convincing even to the writer, who knows that many of them are not in accord with the facts. Coming from a man of Mr. Davis' prominence, they will doubtless carry conviction to many. It would be tedious and appear contentious to answer his discussion item by item, but certain additional facts need presentation and certain of his inferences and misinterpretations of the writer's position need correction.

Under the heading "Storage in Upper Basin" (page 384), Mr. Davis discards the Ouray Dam site on account of a railroad right of way granted by the Secretary of the Interior. The right of way in question was granted in January, 1922, for a period of five years to the Denver and Salt Lake Railroad Company on Mr. Davis' recommendation. It lies entirely on the west side of the river, extending from about the middle of Sec. 18, T. 8 S., R. 20 E., to the Town of Randlett, and is all above the 4 800-ft. contour. Information is not available as to where the proposed railroad will cross the Green River, but there seems to be no reason why it should not follow up the river and cross just south of Split Mountain, somewhat on the line of a right of way granted some years ago. This latter right of way has now expired so that the location of the proposed railroad in the vicinity of Green River is still under Federal control. The water surface at the Ouray site is at about Elevation 4 630. Taken in conjunction with Flaming Gorge and Juniper a storage capacity of about 3 000 000 acre-ft. at Ouray will give full regulation of the Green River at the latter point. This capacity can be obtained by a dam 170 ft. high, that is, with a crest at Elevation 4 800 and a draw-down at 25 ft. A dam of about that height backs the water to the Split Mountain Dam site for the development of which there are now two applications before the Federal Power Commission; it is unlikely that a higher dam at Ouray will ever be built. The Salt Lake Railroad may be a desirable development, but should not be a reason for interfering with a proper development of the Ouray Dam site.

Although up-river storage cannot be used exclusively for carry-over storage for the Lower Basin to the extent at one time contemplated by the Reclamation Service, it will provide seasonal regulation and can be developed for carry-over storage to whatever extent the future may show to be economically feasible. Mr. Davis states that at least 25 000 000 acre-ft. of storage will be needed in the Colorado Basin. In this figure, he duplicates quantities for flood storage and adds a margin for silt storage, but gives no credit for the silt storage that will be provided by power dams. About 86% of the water reaching Boulder Canyon comes from above Glen Canyon, and considering the possibility of storage dams at Glen Canyon and Cataract Canyon in addition to those higher up, this part of the flow can be completely regulated above Grand Canyon. Such regulation can proceed gradually as economic conditions justify and the extent to which it should be developed can be determined by the economic requirements of the future. It will be relatively free from silt deterioration and will have several times the power value of storage at Boulder Canyon. About one-half the remaining flow is a nearly constant increment requiring no regulation, the remainder occurring in erratic flood

flows mainly due to fall and winter rains and rarely exceeding 1 500 000 acre-ft. in any year. These flood flows can be largely regulated and absorbed by the power dams in the Canyon Section and the re-regulating dam below. Such being the case the writer can see no justification for creating 34 000 000 acre-ft. of storage at the bottom of the Canyon Section.

The Reclamation Service estimates indicate that the Big Boulder Canyon project is cheaper for power and storage than any other on the river. This may or may not be the case, but even if it be assumed that it is, the project should not be built unless it is made to conform to a plan of development that will get the fullest use of the water. Colorado River power can be developed cheaper than steam power in Southern California. It will all be needed and nobody should be permitted to "skim the cream," leaving a residue that cannot be used.

Referring to use of up-river storage, Mr. Davis asserts that "This program would be an economic blunder of the first magnitude." His views on economics have often differed from those of the writer, but never more than on this point. It is undoubtedly feasible to regulate the flow of the Colorado above Glen Canyon. The advantage of placing regulatory storage as far up stream as possible is generally recognized, and on no other river known to the writer are the advantages so marked as on the Colorado.

Mr. Davis and most of the Boulder advocates disregard the time element in discussing the Colorado. They compare the initial development of a progressive complete project with the very comprehensive results claimed for Boulder Canyon. If only one development were to be made on the Colorado, their arguments should be given considerable weight, but that is not the case, and Mr. Davis himself recognizes that the development of power will proceed rapidly and continuously to its full extent when the present deadlock is broken.

Mr. Davis' statement that, "it is certainly better to hold floods by storage, even if part of the water does evaporate, than to let them run to the sea, and fight them en route, as demanded by Colonel Kelly," is the kind of argument advanced by several of the Boulder advocates. The writer has never demanded any such thing; on the contrary, he has endeavored to ascertain what degree of flood protection can be obtained by storage, and has indicated what he believes to be profitable as a first step, with the definite statement that full regulation will be obtained later as power and irrigation development proceeds.

Under the heading, "Mohave Reservoir Wasteful and Destructive" (page 386), Mr. Davis exaggerates all the difficulties and slights all the advantages. The estimate of the Reclamation Service for flowage damages on the Mohave site is $12 775 000, made up as follows:

Railroad	$8 500 000
Highway	200 000
General property damage	4 075 000

The railroad damages are based on a letter from an official of the railroad company which bears evidence of an intention to play safe. No allowance is made for the elimination of heavy grades on the present approaches to

the bridge and the extra operating costs due to the additional three miles of road are capitalized at about $1 000 000. No allowance is made for salvage on the present division shops, ice plants, and other property. On the other hand, the estimate of flowage damages at the Boulder site, the development of which would cause the flooding of many mining claims, two small towns, and a branch line railroad terminus, is only $500 000. Evidently these two estimates are not made on the same basis and are not comparable.

With respect to irrigable land in Mohave Valley, it should be understood that the river meanders through this Valley in such a way that under present conditions the cost of reclaiming the land by holding the river in place is too great to make the land of any value. What the conditions will be when the river is controlled to a maximum flow of 40 000 cu. ft. per sec., as proposed by the Reclamation Service, is problematical. It should be less expensive to reclaim the land under those conditions, but levees and bank protection will still be necessary, and will cost at least $50 000 per mile. The land is divided by the river into five tracts, as shown in Table 35.

To the cost of reclamation must be added the cost of irrigation and drainage, which will probably be not less than $75 per acre. To the usual operating costs must be added a figure for maintenance of levees and bank protection. Considering these facts, none of this land is economically feasible for irrigation, with the possible exception of the tract at the south end of the Valley. A dam at Parker of the height proposed by Mr. Davis would submerge this tract; a dam at Parker that does not submerge this tract will not materially interfere with a dam at Mohave.

TABLE 35.—Irrigable Lands, Cottonwood and Mohave Valleys.

(Acreages from Unpublished Report of U. S. Reclamation Service, 1916. Other Figures Compiled by the Writer.)

Tract.	Area, in Acres.						Miles of revetment and levee.	Estimated cost per acre of levee and revetment.	Remarks.
	Irrigable.		Waste.		Total.				
	Gravity.	Pump.	Gravity.	Pump.	Gravity.	Pump.			
Cottonwood Valley:									
East side	2 085	.....	1 551	.....	3 636	.....	5	$125.00	.....
West side	2 214	.....	703	.....	2 917	.....	7	160.00	.....
Mohave Valley:									
Arizona side	.....	700	2 200	400	2 200	1 100	..	.....	All gravity land has been ruined by meandering of river.
Opposite from Mohave	1 000	70	4 800	230	5 800	300	10	500.00	.....
Arizona side	22 500	2 425	12 500	1 200	35 000	3 625	28	56.00	.....

Mr. Davis states that engineers who examined the Mohave site twenty years ago rejected it. It may be well to record why this site was then rejected.

The flooding of the railroad, of the Town of Needles, and of 40 000 acres of supposedly irrigable land was believed to be involved in developing a reservoir capacity limited to 1 500 000 acre-ft. On these premises, it was concluded:*

"In view of the existence of several reservoir sites below and of the damage which would occur from the construction of a dam at Blue Canyon (Mohave Canyon), it is believed that this valley near Needles (Mohave Valley) can be more profitably utilized by irrigation from diversion canals (than as a reservoir)."

If the premises were correct one could easily agree with the conclusion, which, as it should be, was based on the greatest measure of profit to be obtained from the resources involved. It was not reported as "infeasible" despite Mr. Davis' statement that it was "unanimously regarded as infeasible." For a reservoir, the Valley is unusually well shaped; the floor is quite flat and the banks rise steeply from it so that it can be used without creating much shallow water. With a dam 180 ft. high reserving 4 000 000 acre-ft. for flood control, the depth of water at the upper end of Mohave Valley at maximum draw-down would be about 90 ft., and at the upper end of Cottonwood Valley about 40 ft. This cannot properly be termed a shallow reservoir.

Under the heading, "Wanton Destruction of Valuable Storage" (page 387), Mr. Davis argues that storage at Boulder Canyon must be reserved for silt. On the basis of 100 000 acre-ft. of silt per year, the smaller Boulder Canyon Dam suggested by the writer would hold the entire deposit for a period of 100 years. The power dams above would hold at least as much more. No sufficient study of the silt problem has yet been made; probably storage will always be the cheapest way of taking care of it. If the silt problem were to control, no dam above Boulder Canyon should be built until Boulder Reservoir had filled to the maximum for comfortable operation. Then the next dam up stream should be built and allowed to fill, and so on. Such a program would run over a period of several hundred years and would be intolerable. Although silt will add to the expense of using Colorado water for domestic, irrigation, and power purposes, it will not prevent that use provided sufficient storage can be maintained to regulate the flow. As nearly all the silt enters the river below the San Juan, storage above that point will be little affected. Ultimately, at least in theory, all storage below the origin of silt will fill, and the only regulation of flow will be that which can be obtained above that origin. The silt problem is a strong argument for seeking regulatory storage above the San Juan, but not for excessive storage at Boulder Canyon. It is quite certain that complete desilting at Boulder will greatly accentuate the ever-present tendency to erode banks and to pick up a heavy load of silt from the vast quantities now stored in the bottoms along the 300 miles between that point and Yuma.

Under the heading, "Proper Plan of Development" (page 387), it is encouraging to note that Mr. Davis gives consideration to adapting his Boulder Canyon project to a full development of the lower river. Perhaps he will eventually come to consider also the upper river and to give proper weight to the regulatory storage that can and will be provided there. The only regulatory

* First Annual Report, U. S. Reclamation Service, pp. 113 and 114.

storage needed in the lower river in the final development is in the lowest reservoir that will give sufficient storage, so that no power will be lost owing to the difference in the power and irrigation demands. Mr. Davis' discussion makes it necessary for the writer to point out again that he is not committed to his plan, and that it was used simply as an illustration. A better plan can doubtless be developed when the necessary data are available.

In discussing "Poor Rock in Glen Canyon" (page 390), Mr. Davis emphasizes the writer's presumption in differing with his eminent Board. The writer questions whether there is much difference between his own views and those of the Board regarding the quality of rock at Glen Canyon. The report of the Board which examined this site was signed by Messrs. F. E. Weymouth, F. L. Ransome, L. C. Hill, and A. J. Wiley (three engineers and a geologist), on December 20, 1922. The following is quoted from the report:

"11. The Jurassic sandstone has been fully described by Gregory, Bryan, and others. It is a fine-grained, very uniform quartzose sandstone which appears to owe its reddish tint to the superficial redness of certain individual grains. The grains are imperfectly cemented and the whole resembles in hardness the type of soft brick known to the trade as salmon brick. It crumbles under shock, such as that of ordinary blasting, and small fragments can be crushed to sand between the fingers. Notwithstanding its softness the rock stands remarkably well in the canyon walls, forming large, smooth cliffs that rise for 1 000 ft. or more above the river, and which in places are within 5° of being vertical.

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"13. As pointed out by Dr. Bryan the Jurassic sandstone is too soft and too easily broken on corners and edges to make good building stone, and is entirely unsuitable for use as concrete constituent. In large masses, however, it successfully resists the weight of the towering canyon walls and shows no signs of failure at the base of the cliffs where these come down to the water's edge. Under the atmospheric conditions prevailing at the canyon, moreover, the sandstone in spite of its softness withstands the action of the weather remarkably well. Some of the smooth walls must have stood without appreciable change for centuries.

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"30. The abutments for a gravity type concrete dam are fully exposed, and the foundation is indicated by a diamond drill boring. Both are of a quite uniform soft sandstone which is not hard enough for a building stone, but probably will be found to have sufficient strength to support a concrete dam."

As far as the writer can ascertain, no other Board report on the subject has been made, but apparently one member of the Board expressed somewhat modified views about a year later, for Mr. Weymouth, in his report of February, 1924, quotes from a letter of November 27, 1923, by Mr. Wiley as follows:

"It does not seem feasible to build any type of masonry dam of the necessary height for effective storage on the soft sandstone at Glen Canyon, at least no type or height requiring maximum pressures of more than 20 tons per square foot should be used."

A few years ago the writer spent four days examining this dam site. Evidence of bearing power is furnished by the cliffs which rise nearly vertical to a height of 1 000 to 1 500 ft. It may be well to point out that bearing power

is determined by compactness, rather than by hardness and that the rock in question leaves nothing to be desired in the way of compactness. The cementing material is not strong enough to resist heavy erosion; spillway and power tunnels would have to be lined, but in foundations, even with the most ordinary precautions to prevent failure by displacement, the rock will certainly bear more than the load which it is safe to place on concrete in a dam.

It may be interesting to note that common practice considered the safe maximum stress in concrete dams as 20 tons per sq. ft., until some of the high dams of the Reclamation Service were built, when it was found that the cost of maintaining that standard was prohibitive, and the limit was raised to 30 tons per sq. ft. The same experience resulted from the design of the high Boulder Dam; Mr. Davis and his Board of Engineers have designed that dam with a maximum stress of 40 tons per sq. ft., equivalent to 555 lb. per sq. in. The concrete will have to be unusually good to insure an ultimate strength of 2 000 lb. per sq. in. In other words, the factor of safety will be about 3.6 in the concrete. The loading assumptions used by the designers are conservative, but with the tremendous loads available for unexpected concentration in such a high dam, there is a risk in so far exceeding precedent which nothing but positive necessity can justify.

The economic height for power dams in the Canyon Section can be shown to lie between 200 and 400 ft. Dams higher than 400 ft. can be justified only for storage or, perhaps, for the purpose of backing the water up to the next feasible dam site, and should not be permitted unless it can be positively proved that the full development of the river cannot be obtained without them. The writer does not consider it necessary or desirable to build a very high dam at Glen Canyon—probably not over 400 ft., and certainly not over 500 ft.

Referring to the heading, "Increase of Low-Water Flow" (page 391), legal views of eminent authorities are available to prove that Mexico can never profit from the storage of water in the United States, but any one who will study the history of the relations of the United States with other countries, especially with those of the American Continent, will realize that the United States always sets aside legal arguments and settles on a basis of unreasonable liberality. It is certainly a wise precaution to obtain a treaty with Mexico to cover the question. Fortunately, conditions seem favorable at present for negotiating such a treaty and it is understood that preliminary steps are already under way.

Mr. Davis makes two statements in his discussion which cannot be overlooked. In the first he says, "This emphasizes the real point at issue, that is, whether the immense resources of the Lower Colorado are to be retained in the control of the Federal Government or turned over to private corporations." Any one familiar with the Federal Water Power Act knows that it sets forth a complete Federal control—the product of more than twelve years' consideration by Congress. Nobody, as far as the writer is informed, has ever proposed less Federal control except certain advocates of municipal development who do not wish to be burdened by the Federal Water Power Act, and who have called forth the private monopoly specter in their efforts to obtain special privileges for themselves.

The other point is where Mr. Davis states:

"The author deserves thanks for the frank official declaration of the settled policy of the Federal Power Commission, in his statement that: 'All the development needed on the Colorado will be built by private capital under adequate Federal and State regulation if the river is given over to development under the Federal Water Power Act' ".

The writer made a simple statement of fact to indicate that no appropriation by Congress is necessary to procure a full and economical development of the Colorado River. In another part of his discussion (page 386), Mr. Davis confirms this, but he proceeds to construe the writer's plain statement of fact into a declaration of the policy of the Federal Power Commission. The policy of the Federal Power Commission with respect to any pertinent subject can always be obtained by application to the Commission or can be deduced from the precedents set up during the past four years. The Commission has administered the Act strictly in accordance with its terms, and all municipal applications have received the preference to which the law entitled them. The only municipal application for power development on the Colorado River is that made by the City of Los Angeles for the Boulder site. The City made it clear to the Commission that it did not wish consideration of its application unless Congress definitely refused to appropriate for the Boulder Project. Judging by precedent, the City of Los Angeles can feel assured that it will get all the preference contemplated by law if and when any application for the Boulder Canyon Section of the river is considered by the Commission. Mr. Davis' statement is an accusation that the members of the Commission have acted in defiance of the law entrusted to their administration. Such an accusation has no place in an engineering discussion. It may be noted that a similar accusation has been made recently by certain municipal ownership advocates who claim that the Commission is not carrying out the law because it has subjected their projects to the same examination as private projects and has not granted the applications "sight unseen". Mr. Davis seems to be convinced that opposition to his project is due entirely to objection to public ownership. That is not the case; the writer's opposition is and always has been based solely on engineering and economic grounds.

Mr. Grunsky emphasizes the need for a treaty with Mexico. He also discusses the division of the resources of the Colorado among the various States and proposes, as a solution, Federal control. The tendency to mould economic laws to fit political boundaries has always been prevalent; it probably will never be entirely overcome, although it is dangerous and has been responsible for most of Europe's troubles. All the developments on the Colorado thus far proposed involve power and occupy public lands, and, therefore, in the absence of new legislation, will come under the Federal Water Power Act, which provides a Federal control that is adequate. The Act does not provide Federal funds for construction, nor does it give the upper States a treaty guaranty of water rights, but, judging by experience under the Act to date, it will give them all the protection they need.

With reference to Mr. Weymouth's discussion, the writer had no intention of misquoting or misrepresenting the data presented by the Reclamation

Service. Table 11 does not purport to be taken from any report of the Bureau of Reclamation. It is quoted from a report by Herman Stabler, M. Am. Soc. C. E., and is a compilation of data made available to him by Mr. Weymouth. Evidently the same data, slightly modified, were compiled in Table 30 by Mr. Weymouth in the report to which he refers. The essential differences between the two tables are explained in the footnotes to Table 11. The text of the footnotes is based on statements in the report made by Mr. Weymouth in February, 1924.

In Table 19, the term, "U. S. Bureau of Reclamation Plan", was used in a general sense as referring to a high dam at Boulder Canyon, and the data used in the table were computed by the writer. There should be no misunderstanding on this point because all the figures are based on up-river regulation which Mr. Weymouth and the Reclamation Service have declined up to the present to consider as available. Mr. Weymouth cites the historical development of the investigation conducted under his direction. Conditions have changed materially since this investigation was commenced. Mr. Weymouth's willingness to consider an ultimate plan of development as evinced in February, 1924, together with the general trend of his discussion, might be taken as an indication that if he had continued in charge of the investigation he might have spread it sufficiently to get the broader scope which the writer deems necessary. About the only material difference between his views and those of the writer is in regard to the feasibility of storage elsewhere than at Boulder Canyon. The data available on storage sites may be sufficient to have convinced Mr. Weymouth that up-river regulation is infeasible, but it has failed to convince many others, including the writer.

Professor Smith points out the interests of the State of Arizona in the Colorado River. These interests have been fully recognized by all Federal authorities. Unfortunately, there is divided opinion in Arizona as to what should be done with its interests so that much delay in getting started on any project for the Colorado is likely to result. Professor Smith discusses the question of water rights. The writer does not pose as an expert on that subject, but whatever the law may be it is certain that representatives of the upper States have persistently refused to accept anything short of an interstate treaty as protection of their future needs. Professor Smith's suggestion that the matter be settled by the Imperial Valley suing the permittees on Flaming Gorge does not seem to be practicable, because Flaming Gorge cannot in any way infringe on the rights, present or future, of the Imperial Valley, and there is no ground on which to base a suit. Professor Smith's suggestions with respect to up-river storage are sound, but as yet no one has shown a practical way of financing a dam at Dewey in advance of a certain amount of power development below. Professor Smith might with justice point out that the same objection exists against the Mohave site. The Federal Government has considered such projects only in connection with the improvement of navigation or under the terms of the Reclamation Act. It is stretching the point rather far to consider that Boulder Canyon or a substitute comes under either category, and it is doubtful whether Congress will set a new precedent by

undertaking any project on the Colorado unless there is a strong public demand and practically no opposition.

The people who might be expected to support a proposition for a flood-control dam at Dewey are supporting Boulder Canyon and opposing everything else. Under the circumstances, the only prospect for early development of up-river storage appears to the writer to be in connection with power development.

Mr. Jarvis' discussion comprises a brief résumé of the Colorado problem and expresses his views on certain phases of the problem.

Mr. L. C. Hill discusses the Glen Canyon site as a power site. The writer has never considered such a development probable. If the Glen Canyon Dam is built it will probably be as a storage dam to regulate the flow for projects lower down. Eventually, it might be feasible to develop power at Glen Canyon, but that is hardly likely to be economical in the early stages. Mr. Hill seems to appreciate the desirability of making all the units fit into a plan for full development. He misunderstands, however, the writer's views with respect to the relation between power and irrigation, namely, that power development will proceed more rapidly than irrigation but not that power should be allowed to curtail irrigation. On the contrary, construction at power sites can and should progress so that it will permit and aid the fullest practicable irrigation development. The writer does not agree with Mr. Hill that storage for annual hold-over will be necessary or advisable at Boulder Canyon.

Mr. Hill states that about 100 000 acres in Mohave and Cottonwood Valleys would be submerged by a dam near Needles. The total area submerged by the highest dam proposed will be about 75 000 acres and, as shown under the writer's reply to Mr. Davis' discussion, of this less than 25 000 acres can ever be irrigated.

In regard to Mr. Hill's computations on evaporation losses in Mohave Valley, it is difficult to determine with any certainty what conditions will be if the flow is regulated to a maximum of 40 000 cu. ft. per sec. The writer's estimate of present evaporation losses is based on the area overflowed by a river stage 10 ft. on the Topock gauge. This stage has been reached in the past few years with flows varying from 35 000 to 60 000 cu. ft. per sec. The writer believes there will be no material reduction in evaporation losses with the flow regulated to 40 000 cu. ft. per sec. Of course, if any of the overflowed area is reclaimed and put to beneficial use, it should be deducted from the area producing present evaporation losses. Under the most optimistic estimates not more than one-third of the overflowed area will warrant the cost of reclamation; the writer does not believe that any of it will.

In regard to flood protection, Mr. Hill's principal argument is that the writer disregards the opinion of those who have personally been combatting the floods of the Colorado. This is not the case—it was because the writer found considerable difference of opinion among those who have been combatting the floods that he deemed it advisable to make a careful study of the situation. That study is presented in the paper. The facts have been approved

by the Reclamation Service engineers, to whose opinions Mr. Hill doubtless refers, and the writer has found no reason to change his conclusions.

The writer regrets that he did not specifically acknowledge the use of the data prepared by Mr. R. A. Hill. It was obtained from the Bureau of Reclamation, together with much other data, and so was acknowledged *en masse* as the work of that Bureau.

The writer agrees that it will be desirable eventually to equalize the flow of the river as far as practicable, but he also believes that the partial equalization set forth in his paper is sufficient to remove the flood menace and that until certain other matters are arranged, such as a treaty with Mexico and flood control on the Gila, no greater equalization is advisable.

The question has already been discussed as to whether the water supply from the Colorado is sufficient to meet all future irrigation needs, and the writer will do no more than reiterate his statement that in such a vast arid region, whatever present expectations may be, it is certainly prudent to avoid all unnecessary waste of water.

Mr. Lane raises the interesting question as to what results will follow the release of practically clear water into the silt-filled valley below the Canyon Section of the river. There are so many unknown factors in the problem that, as he points out, no one can predict results with certainty. It can hardly be doubted, however, that the river will pick up a new load of silt in a relatively short distance below the dam. Until it has acquired its new load, there will doubtless be an increase in the tendency to meander through the wider stretches of the valley, combined with a tendency to lower the bed of the stream, and these actions may affect the equilibrium of the stream all the way to its mouth.

H. T. Cory, M. Am. Soc. C. E., has discussed* the effect of Laguna Dam, which had a capacity for storing about 20 000 acre-ft. when it was first completed in 1909. Unfortunately, the effect of this silt storage was largely obscured by the effect of change in river conditions below. The river dropped out of its old channel into Bee River and Volcano Lake in 1909 and the lowering of the bed due thereto was probably greater than that due to the clearing of the water.

Mr. Debler's discussion is that of an advocate. He presents no new facts but puts forward arguments to prove that the large Boulder Canyon Dam should be built by the Federal Government. In some instances his statements are not sufficiently complete to give a true impression of the facts. For example, he quotes Messrs. Preston and Priest as saying that "overflow of the natural banks below Yuma begins at discharges of 30 000 to 50 000 sec-ft., depending on river conditions". This statement is correct, but needs qualification. As shown in the paper (page 317), overflow at these discharges does not occur above the lower end of the Yuma Project where the levees on the two sides begin to recede. The writer's original discussion of flood control and river conditions under different discharges was very carefully prepared and nothing in the discussions presented has indicated the need of a change therein.

* *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

One of the chief reasons for considering a development at Mohave Valley is that it is possibly the cheapest place to obtain a strictly flood-control project which would not increase the usable water supply in Mexico.

The writer's presentation of possible power development at Mohave Valley (page 346), was intended to show that the project could be built to have a power value comparable to prospective Southern California projects; the comparison is believed to be essentially fair, but the writer does not advocate development of power at that site before a satisfactory treaty with Mexico has been arranged.

Mr. Debler's statement in regard to development on the Colorado by private capital under adequate Federal and State regulation, that "speedy development can be obtained in this way, along lines, however, as favorable as possible for power but with a minimum regard for irrigation and flood control", implies that power development is in the habit of over-riding irrigation. A review of the power developments that have been made in the arid States will show that this is not the case. Irrigation has not only been fully protected, but has generally been benefited and has rarely paid for the benefits it has received.

As Mr. Debler notes, the writer presents no definite plan of development as a substitute for the Boulder Canyon plan. The Bureau of Reclamation has conducted all investigations made by the Federal Government looking to development on the Colorado. Those investigations were concentrated on the Boulder Canyon Project and did not produce the data necessary to design that project, or any other, so as to conform to a full utilization of the waters of the Colorado. The writer has pointed out this fact and urged action to get the additional information without further delay. The additional information most essential is examination of at least four and preferably seven dam sites with sufficient borings to determine foundation conditions. Given that information, a specific plan can be prepared without much delay. Until that information is available no specific plan can be presented.

These conclusions may be summarized, as follows:

During the past four years as Chief Engineer of the Federal Power Commission, the writer's time has been largely devoted to examining applications for water-power projects to see that "the plans for same * * * shall be such as in the judgment of the Commission will be best adapted to a comprehensive scheme of improvement and utilization for the purposes of navigation, of water-power development, and of other beneficial public uses." As planned by its advocates, the Boulder Canyon Project, with its 605-ft. dam, does not, in his opinion, meet this requirement. Additional field investigations of dam sites immediately above and below Boulder Canyon are necessary to determine with certainty the extent to which the Boulder Canyon Project should be modified to make it fit into the best plan of development.

The re-election in 1924 of Governor Hunt, of Arizona, on a platform definitely opposing the Colorado River Compact indicates that early ratification of that Compact is unlikely. The most active proponents of the Boulder project are primarily interested in securing a large block of power. The

project proposed has the physical capacity for appropriating and using more water than the Compact allots to the Lower Basin. Its advocates are not willing to curtail the capacity, and their proposals for legislative limitation have been rejected by the representatives of the upper States. Consequently, in the absence of the Compact the prospects of a compromise between advocates of the Boulder Canyon Project and the upper State interests do not look promising. It seems highly desirable to make the most of this probable delay by completing the field investigations needed and by negotiating a treaty with Mexico.

It is regrettable that the beginning of development on the Colorado should face such a delay, and the writer has given much thought to ways and means of avoiding it. It would seem that the upper States would have no logical ground for objection to any project that is designed and carried out on plans that would prevent the appropriation and use of more water than is allotted to the Lower Basin by the Compact. The Mohave Valley site was suggested as a flood-protection project on this basis. Perhaps the Dewey, combined with some other up-stream reservoir, would serve as well. To date, however, the interests needing flood protection have shown no disposition to look for any solution other than Boulder Canyon.

The Flaming Gorge and Diamond Creek projects are both desirable. Neither will conflict in any way with the terms of the proposed Compact. Although they will not, in themselves, greatly benefit the Lower Basin, they constitute a first step toward the ultimate complete development of the river. Both are authorized under preliminary permits issued by the Federal Power Commission; they will ask for licenses to begin construction in the near future. It is some time since the Federal Power Commission has given these projects consideration, and the writer cannot predict what action it will take, but all things considered, it appears that they offer perhaps the most favorable opportunity for a start on developing the Colorado.

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Paper No. 1559

THE INTERCONNECTED POWER SYSTEMS OF THE SOUTHEAST*

By Charles G. Adsit,† M. Am. Soc. C. E.

With Discussion by Messrs. Joseph Hyde Pratt, William P. Hammond, Dana M. Wood, George A. Orrok, E. A. Yates, and Edgar A. Van Deusen.

Synopsis

Many articles have appeared in the press and in scientific magazines during recent years on the super-power systems of the country, and much attention has been directed to what has been called the super-power system of the Southeastern States, as illustrating one of the best developed power systems in the United States. Some of these articles have discussed at great length the question of voltage control, switching troubles, and other operating difficulties to be encountered and solved. Others have contained actual set-ups for conditions five to ten years hence, with maps of new lines which should be built, and have included tables showing the vast saving which will annually result in the operation of such a system, such saving being the direct result of improved load factor, concentration of generating capacity in large plants at favorable locations, and utilization to the maximum possible extent of all available water power, even in times of high-water or flood conditions. The saving, expressed in dollars, under some of these plans is so great that there is a marked tendency to become over-enthusiastic as to the immediate need and possibility of the proposed system, and to forget the physical and financial problems involved in making predictions or drawing conclusions.

It should be remembered that the growth of a generating and transmission system in any section of the country is distinctly evolutionary in character, and

* Presented at the meeting of the Power Division, Atlanta, Ga., April 10, 1924.

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that the interconnected transmission systems of the five States of the Southeast, which to-day present the nearest approach to what many writers mis-name a "super-power system" (with, perhaps, the one exception of the systems of California), were not built as a super-power system, but as an interconnection of various high-tension systems already in existence, so that the maximum interchange of power could take place between these systems as a whole, operated from a purely economical standpoint. It exists to-day as a group of high-voltage transmission lines and generating plants which were begun as small units and were rapidly expanded until the distributing lines were nearly touching each other at various points, and the actual interconnection involved the construction of connecting lines of relatively short length. It is thus apparent that the interconnection of these various systems was not the prime consideration in their development, but a logical and purely secondary step. In some cases, the actual connection, rather than calling for additional expenditures, had to be regarded as a credit item, in that it obviated the necessity of duplicate or overlapping distributing circuits. In only a few cases have the connecting lines been built for their actual value as an interchange power connection between the two companies, without any consideration of the economic features of the proposed extensions based on the new loads to be served thereby.

These statements have been made with no desire to throw cold water on the various so-called super-power systems which have been proposed for different sections of the country, nor any lack of appreciation of the service which is being rendered the power industry by the various studies which engineers are devoting to this subject, but simply to suggest that such development as does take place along these lines in the next decade will be a logical one in which each step taken will be justified by the economic considerations at the time rather than the economies which will prevail when the completed system has been perfected.

The Georgia Railway and Power Company's System described herein is not the largest in the interconnected transmission systems of the Southeast, but its growth and plan for future extensions are typical of the other companies connected therewith.

In 1902 and 1903, the first water-power plant in this section was completed at Morgan Falls, on the Chattahoochee River, 16 miles from Atlanta, Ga., and a 22 000-volt line was built from this point to Atlanta by a company known as the Atlanta Water Power and Electric Company. This plant consists of a 48-ft., solid masonry dam about 1 000 ft. long, and a power-house which contained seven 2 500-h.p., horizontal type, water-wheels, operating under a head of 48 ft., direct-connected to seven 1 500-kw., 3-phase 25-cycle, alternating current generators, having a total capacity of 10 500 kw., with the necessary transformers for stepping up the generating voltage of 2 300 to 22 000 volts. It was built for operation at 25 cycles, and had an annual rated output of approximately 30 000 000 kw-hr., provided the load could absorb this amount of power. It was regarded at that time as a very large hydro-electric development, and many famous engineers were consulted before its construction was undertaken,

the names of whom appear on a bronze tablet in the plant. This plant is now being re-built as a 60-cycle plant, with a total installation of 15 000 kw. in generating equipment and 22 500 h.p. in water-wheels, by rewinding the generators and replacing the old water-wheels with new units of more modern design. Mention of this plant is made as being one of the first important water-power plants built in the South, and also in order to show the difference in output of a plant operated on a separately connected load, with the same plant connected to a larger system, such as that under consideration. The Morgan Falls plant was built with an expected output of 30 000 000 kw-hr., but after changing it to 60-cycle, and connecting it to this large power system, the annual output has become more than 60 000 000 kw-hr.

From 1902 to 1912, the whole Southeast, including the City of Atlanta, enjoyed a rapid growth, and the distributing lines around the city were extended from time to time to supply the increased demands of the industrial and lighting loads, but the real impetus in the power development of the State came in the few years prior to 1912 with the organization of the Georgia Power Company, when it acquired the necessary lands and water rights and proceeded to develop the power in the Tallulah River, at Tallulah Falls, Ga. The proposed plant was to operate under a head of 608 ft. and have an immediate installation of 60 000 kw., with space provided for the installation of an additional 12 000-kw. unit at a later date. The development attracted much attention on account of the high head involved, which at that time was the highest head utilized with any plant east of the Rocky Mountains, and on account of the local opposition which was encountered in prosecuting the work. For many years prior to this date Tallulah Falls had been regarded as one of the wonder spots of Nature, where the continual roar of the water passing over the Falls, the gorgeous mountain scenery of the section, and the delightful summer climate attracted many visitors each summer, and the idea of taking the water from the river and passing it silently through a large tunnel, penstocks, and water-wheels, to render a purely commercial service, rather than allow it to continue its natural noisy course from rock to rock over the precipitous falls at the head of the gorge, appeared to many to be little short of desecration, and some people, including the State authorities, spent large sums of money in attempting to prevent the development. The reader may laugh at this, forgetting that it was necessary to mount cannon on some of the first railroad trains to protect the trainmen from injury at the hands of people in the country through which it passed, and that practically every forward step in the world's progress has been bitterly opposed when first suggested.

It should be added at this point, however, that such opinions are not now generally held, and that even the people who most bitterly opposed the Tallulah project on the grounds that it would destroy Nature's handiwork are now convinced that the water of the Tallulah River is performing a greater service to mankind than before the development, as the power generated by this water is transmitted all over the State to help in lighting the homes of thousands of people, whereas before it could only be viewed or heard by a few who were

fortunate enough to spend a holiday season at Tallulah Falls. Furthermore, it is generally agreed now that the mountain lakes which have been formed by the various dams above the Falls are equally as magnificent and have meant more to the immediate development of the surrounding country than the original Falls themselves.

The total generating capacity of the Georgia Railway and Power Company on January 1, 1914, was as follows:

	Kilowatts.
Tallulah Falls Plant (Hydro)	60 000
Morgan Falls Plant (Hydro)	10 500
Dunlap Plant (Hydro)	2 200
Butler Street Steam Plant, Atlanta	10 000
Davis Street Steam Plant, Atlanta	10 000
Total	92 700

The transmission system of the Company then consisted of two 110 000-volt circuits strung along a steel tower line from Tallulah Falls Station to Atlanta, 90 miles, a 50 000-volt, single circuit line on steel towers from the Dunlap plant near Gainesville, Ga., to Atlanta, about 50 miles distant, and two 22 000-volt circuits on wood pole lines from the Morgan Falls plant to Atlanta, a distance of 16 miles. There was also one 110 000-volt circuit, 71 miles long, supported by steel towers between Atlanta and Lindale, Ga., for supplying the Lindale Station and intermediate points along this line, and one 110 000-volt circuit from Atlanta to Newnan, Ga., also supported by steel towers, a distance of 40 miles, these steel tower lines having been constructed in 1912 and 1913, as a part of the Tallulah Development. Transformer stations, of the most modern design, with capacities of 3 000 to 30 000 kw., were also erected to step down the voltage from 110 000 to 22 000 or 11 000 volts at Lindale, Marietta, Cartersville, Newnan, Gainesville, and Atlanta, Ga., for supplying the low-voltage distribution circuits to be extended from these points as the demand for power materialized.

The anticipated output of the Tallulah Falls plant at the time it was constructed was approximately 125 000 000 kw-hr. per annum, but the connected load on the Company's system at that time could not absorb this power, so a contract was negotiated with the Southern Power Company for a part of the output, and a steel tower line was built from Tallulah to Easley, S. C., supporting two 2/0 circuits, each good for a capacity of 20 000 kw., for delivering a part of the output from the new plant to the Southern Power System.

This 60-mile line, with a capacity of approximately 40 000 kw., was the first step in the interconnected system as it now stands, connecting the system of the Georgia Railway and Power Company with that of the Southern Power Company.

Following this initial high-voltage line construction, just described, arrangements were made for constructing a 1/0 copper circuit on wood poles from the Tennessee System at Cleveland, Tenn., into the northern part of the State of Georgia, to serve the territory around Dalton and Calhoun, and this line was finally extended to Lindale, to connect with the Georgia System

at this point to serve as a tie line between the two systems as well as a distributing feeder for the intervening territory. It was designed for and is still operated at 60 000 volts, with a capacity of approximately 10 000 kw. The usefulness of this line was rapidly outgrown. A new 4/0, 110 000-volt, single circuit line was started in 1923 to connect Lindale with the Tennessee Power System at Chattanooga, Tenn., this line to have a capacity of 25 000 to 30 000 kw., in addition to the 60 000-volt line.

The Central Georgia Power Company, operating in the central part of the State around Macon, Ga., also soon afterward extended a 66 000-volt double circuit on steel towers from its new water-power plant at Lloyd Shoals, through Griffin, Ga., to the outskirts of Atlanta, where connection is made with the Georgia Railway and Power Company's System at a station known as the South Atlanta Station.

The Columbus Power Company, some years later, extended its transmission system from its water-power development at Goat Rock, near Columbus, Ga., to Newnan, to supply cotton-mill loads in the Newnan territory, and a short line about three miles long was next built between the sub-station of this and that of the Georgia Railway and Power Company at Newnan, for the purpose of tying these two generating systems together, and permitting the interchange of power between them. A daily interchange has taken place between the Columbus Power Company and the Georgia Railway and Power Company since this interconnection was made in 1914. This has resulted in an immense saving of power to the State, the Georgia Company taking all the excess hydro-power during nights, Sundays, and holidays, that the Columbus Power Company has had available, and returning it to the Columbus Power Company during its heavy load hours in the day.

In 1920, plans which had been under way for some time were finally consummated, and construction was begun on a 110 000-volt, single circuit tie line between the Gadsden Steam Station of the Alabama Power Company's System and the Lindale Station of the Georgia Railway and Power Company, which was completed and put under voltage during 1921.

On the completion of the Alabama tie line referred to, in 1921, the Georgia Railway and Power Company's transmission system became the central unit of a group of seven large power-generating and transmission companies, connecting on the north with the Tennessee Power Company, on the east with the Southern Power Company and the Carolina Light and Power Company, on the south with the Central Georgia Transmission Company and the Columbus Power Company, and on the west with the Alabama Power Company, by means of which the power generated on any of these systems can be delivered to the distribution lines of any of the other six systems.

The Georgia Company still operates the various small hydro-plants acquired and the steam plants at Atlanta, but its principal source of energy is the water power of the Tallulah and Tugalo Rivers, which will next be discussed in some detail.

The Tallulah and Chattooga Rivers unite to form the Tugalo, as shown on Fig. 1, and the latter joins the Seneca to form the Savannah, which forms the boundary line between Georgia and South Carolina. These rivers are fed by

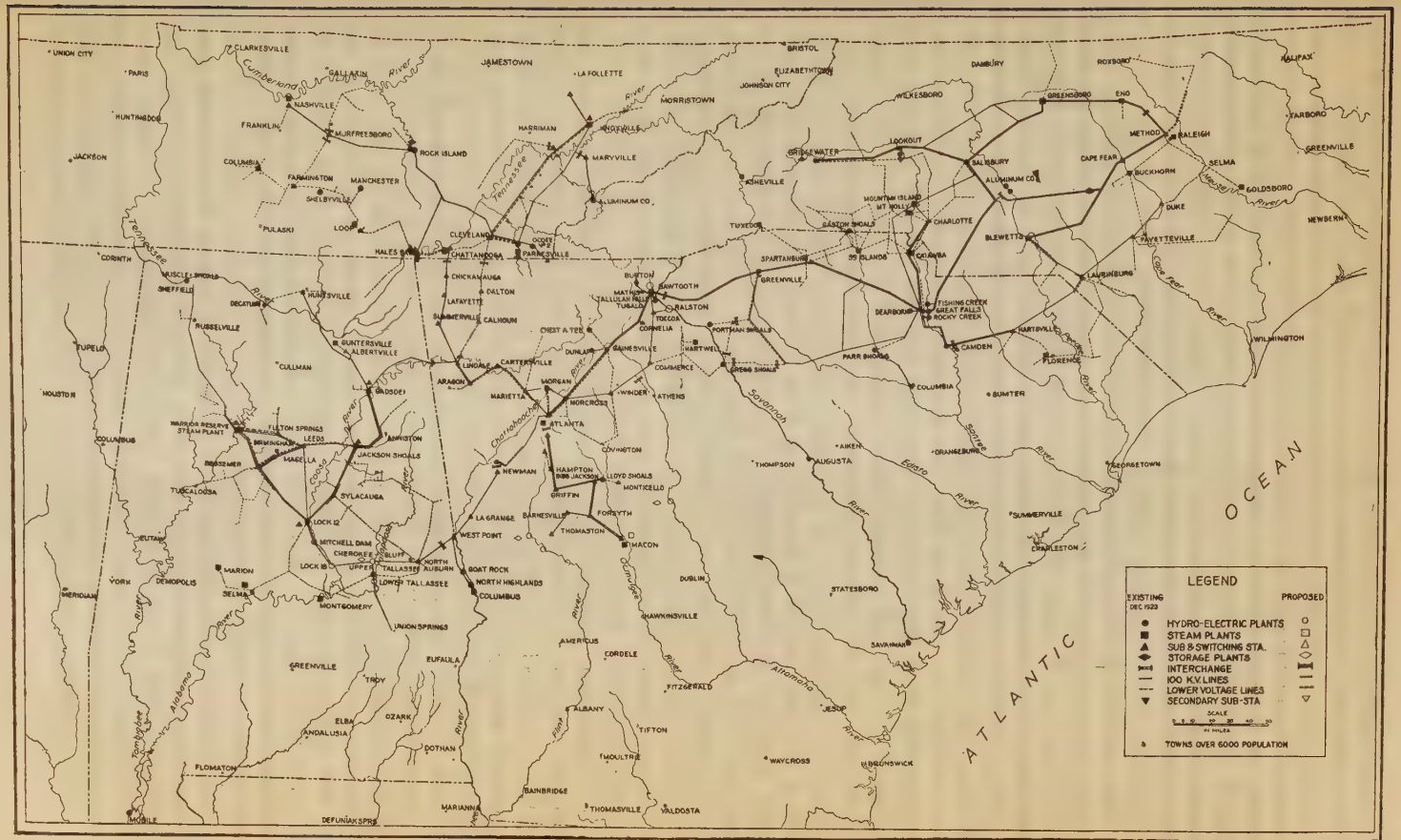
LEGEND
EXISTING DEC 1929
PROPOSED
HYDRO-ELECTRIC PLANTS
STEAM PLANTS
SUB & SWITCHING STA.
STORAGE PLANTS
INTERCHANGE
100 K.V. LINES
LOWER VOLTAGE LINES
SECONDARY SUB-STA
SCALE
IN MILES
TOWNS OVER 6000 POPULATION
ATLANTIC OCEAN
UNION CITY
PARIS
HUNTINGDON
JACKSON
CORINTH
TUPELO
HOUSTON
COLUMBUS
STARKVILLE
MERIDIAN
YORK
DEMOPOLIS
SELMA
MONTGOMERY
UNION SPRINGS
TROY
GREENVILLE
ELBA
ANDALUSIA
FLOMATON
DEFUNIAK SPRS
MOBILE
EUTAW
PULASKI
COLUMBIA
FARMINGTON
SHELBYVILLE
FRANKLIN
NASHVILLE
MURFREESBORO
CLARKSVILLE
GALLATIN
JAMESTOWN
ROCK ISLAND
MANCHESTER
LOOP
HALES BAR
CLEVELAND
CHATTANOOGA
ODDEN
PARKSVILLE
HARRIMAN
KNOXVILLE
MARYVILLE
ALUMINUM CO
LA FOLLETTE
MORRISTOWN
BRISTOL
ELIZABETHTON
JOHNSON CITY
ASHEVILLE
TUXEDO
GLENWOOD
LOOKOUT
MOUNTAIN ISLAND
MT HOLLY
EASTON SHOALS
SPARTANBURG
GREENVILLE
TALLULAH FALLS
TUGALO
BURTON
MATHIS
TOCCOA
CORNELIA
TALBOTT
SANTOOTH
CHEST A TEE
DUNLAP
BARNESVILLE
HARTWELL
GREGG SHOALS
PORTMAN SHOALS
SUMMERVILLE
CALHOUN
LAFAYETTE
DALTON
CHICKAMAUGA
LINDALE
CARTERSVILLE
ARABON
MORGAN
MARIETTA
NORCROSS
ATLANTA
WINDER
ATHENS
COMMERCE
GUNTERSVILLE
ALBERTVILLE
RUSSELLVILLE
DECATUR
HUNTSVILLE
SHEFFIELD
MUSCLE SHOALS
CULLMAN
GADSDEN
WARRIOR RESERVE STEAM PLANT
FULTON SPRINGS
BIRMINGHAM
LEEDS
ANNISTON
JACKSON SHOALS
MASELLA
BESSEMER
TUSCALOOSA
SYLACAUGA
LOCK 12
LOCK 13
MITCHELL DAM
CHEROKEE BLUFF
UPPER TALLASSEE
LOWER TALLASSEE
NORTH AUBURN
MARION
SELMA
NEWMAN
HAMPTON
RIDE JACKSON
GRIFFIN
LLOYD SHOALS
MONTICELLO
COVINGTON
LA GRANGE
BARNESVILLE
FORSYTH
THOMASTON
MACON
WEST POINT
BOAT ROCK
NORTH HIGHLANDS
COLUMBUS
AMERICUS
CORDELE
FITZGERALD
ALBANY
TIFTON
MOULTRIE
BAINBRIDGE
MARIANNA
THOMASVILLE
VALDOSTA
WAYCROSS
DUBLIN
HAWKINSVILLE
ELFAULA
OZARK
DOTHAN
BIRMINGHAM
ALUMINUM CO
BLEWETT
CHARLOTTE
CAMDEN
FISHING CREEK
GREAT FALLS
ROCKY CREEK
BEARDSOWN
PARR SHOALS
COLUMBIA
AIKEN
AUGUSTA
THOMPSON
ORANGEBURG
SUMTER
FLORENCE
HARTSVILLE
DARLINGTON
GEORGETOWN
CHARLESTON
STATESBORO
SAVANNAH
JESUP
BRUNSWICK
DANBURY
ROXBORO
HALIFAX
WILKESBORO
GREENSBORO
ENO
RALEIGH
METHOD
YARBORO
GREENVILLE
SALISBURY
CAPE FEAR
BUCKHORN
SELMA
GOLDSBORO
NEWBERN
FAYETTEVILLE
LAURINBURG
WILMINGTON
DUKE
Tennessee River
Cumberland River
Cahaba River
Coosa River
Chattahoochee River
Savannah River
Ocmulgee River
Flint River
Altamaha River
Ogeechee River
Santee River
Edisto River
Pee Dee River
Cape Fear River
Yadkin River
Alabama River
Tombigbee River

FIG. 1.

the many small streams draining the eastern slope of the Blue Ridge Mountains, where the altitude varies from 750 to 3 000 ft. above sea level. The territory surrounding these head-waters enjoys an annual precipitation on the average of approximately 70 in., a figure exceeded by few sections of the United States. It is mountainous in character, and densely wooded, with an annual run-off equal to approximately 60% of the rainfall. The fall in the streams is so great as to make them unsuited for any purpose except power development, and the region is so sparsely settled that little difficulties have been encountered in acquiring the land necessary for such development.

The original Tallulah Development previously mentioned consists of a cyclopean masonry dam, 116 ft. high and approximately 400 ft. long at the crest, built across the Tallulah River immediately above the well-known Tallulah Falls (Fig. 2). It serves to divert the water of the Tallulah River to an intake structure, built on the right bank of the stream a short distance up stream from the dam. The intake structure is a heavily reinforced, self-contained concrete tank, 40 by 80 ft., and 50 ft. deep, below normal water level, which contains the necessary gates and operating equipment, and the racks or screens which prevent débris or other material from passing into the tunnel. The tunnel which leads from the intake structure to the surge tank on the side of the gorge above the power-house is horse-shoe shape in cross-section, about 11 ft. wide at the base, and 14 ft. high. It is 6 666 ft. long, cut through solid rock, and is lined with concrete throughout its entire length, the slope of the tunnel being only 2 ft. per 1 000 ft. It discharges into the surge tank, another reinforced concrete structure, 30 by 70 ft. and 95 ft. deep, located at the top of the gorge above the power-house, from which the water is passed to the wheels in the power-house below through six steel penstocks set on concrete piers down the steep side of the gorge, each about 1 200 ft. long.

The power-house consists of two main divisions, the generator building and switching-equipment building, both being built with concrete substructure and red brick and steel superstructure, capped with tile roofing. The generating equipment consists of six 18 000-h.p., water turbines of the reaction type, direct-connected to 12 000-kw., vertical type generators. These machines generate at 6 600 volts, 3-phase, 60-cycle, and the power is stepped up to 110 000 volts for delivery to the outgoing lines by six transformer banks of equivalent capacity. The plant operates under a net head at full load of approximately 590 ft., the gross head under static water conditions being 608 ft. The annual output of the plant with only such storage as is provided by the small lake above the diverting dam was estimated to be approximately 125 000 000 kw-hr. The first five units were installed in 1912 and 1913, and the sixth unit was added in 1917 and 1918.

The second development on the Tallulah River System was made in 1913 and 1914, and consisted of the Mathis Dam and Reservoir (Fig. 2). This dam is about 6 miles, by river distance, above Tallulah Dam, or 5 miles above the back-water of Tallulah Lake. It is built of reinforced concrete, and is approximately 90 ft. high and 700 ft. long, and forms a lake flooding 834 acres, and holding 1 369 000 000 cu. ft. of water, which is equivalent to

15 000 000 kw-hr. at the Tallulah Falls plant. It was constructed to form a reservoir to regulate the river flow above the Tallulah Falls plant, and performed its purpose to the extent of increasing the annual output of the Tallulah plant to approximately 140 000 000 kw-hr. per annum. By 1916, the load on the Georgia System was increasing at such a rate that additional developments were required, and, in 1917, after careful analysis of many available sites, plans were consummated for the immediate construction of a second large storage reservoir above the Tallulah plant to be called the Burton Reservoir, and for a new dam and power plant on the Tugalo River to be known as the Tugalo Development, immediately below the junction of the Tallulah and Chattooga, which could utilize the water passing the Tallulah plant, in conjunction with the unregulated flow of the Chattooga.

The Burton Dam was soon under construction, and was rushed to completion in 1918. It is a solid cyclopean masonry dam 116 ft. high, about 1 250 ft. long across the crest, and contains 112 500 cu. yd. of masonry. It forms a

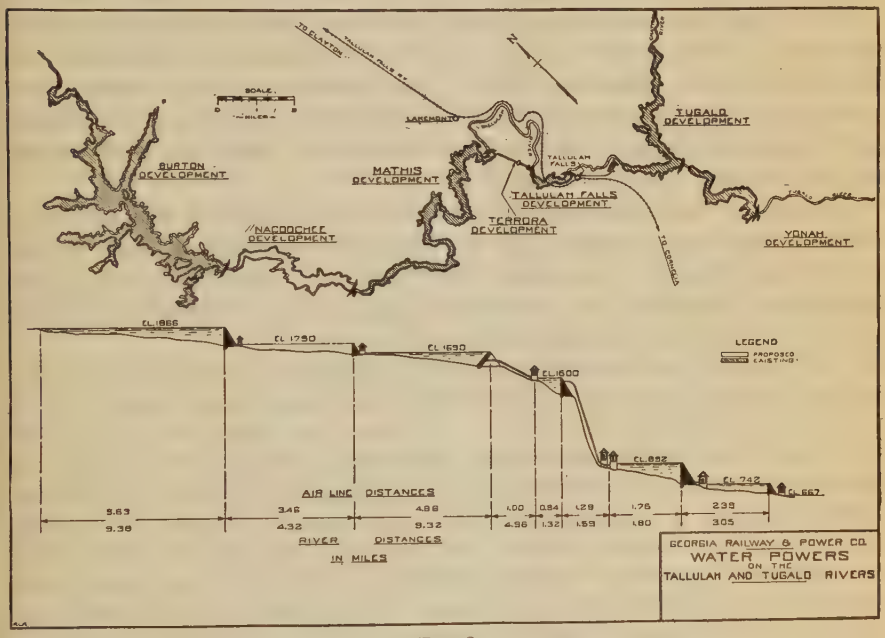

FIG. 2.

reservoir which covers 2 775 acres and holds more than 5 000 000 000 cu. ft. of water, equivalent to 55 000 000 kw-hr. at the Tallulah plant. Its construction was justified by a study of hydrographs which showed that, during the spring and winter months, the flood waters of the Tallulah River that could not be retained by the existing reservoirs, were sufficient to fill the Burton Reservoir four years out of each five. The operation of the Tallulah River plants since the construction of this dam have proved that this reservoir has increased the annual output of the Tallulah plant approximately 50 000 000

kw-hr. per annum, making the present rating of the plant 190 000 000 kw-hr. in a year of normal water flow, with more than 200 000 000 kw-hr. in years of heavy rainfall, these reservoirs regulating the flow of the Tallulah River to the extent of making 90% of the annual run-off available for power generation.

On account of the unsettled financial and labor conditions following 1917, the Tugalo Development had to be postponed, but was resumed in January, 1922. This development consists of a cyclopean masonry dam, 150 ft. high, and approximately 1 000 ft. long, containing 225 000 cu. yd. of masonry, together with a power-house and transformer station with 50 000 kw. installed in generating and transforming equipment, and 88 000 h.p. in water-wheels. The plant operates under a head of 150 ft., formed entirely by the dam, and has a rated output of 140 000 000 kw-hr. under normal water conditions. As it is below the junction of the Tallulah and Chattooga Rivers, on the Tugalo, it will utilize all the water passing through the Tallulah Falls plant, regulated by the three reservoirs above, together with the unregulated flow of the Chattooga River, a stream approximately twice the size of the Tallulah. The lake formed by the dam itself has an area of 557 acres, and contains an available draw-down of 500 000 000 cu. ft., which can be utilized for regulating the 24-hour flow of the Chattooga River. The plant contains four units of the vertical type, similar to the machines in the Tallulah plant, two of which were put in service in October, 1923, and the other two of which will be in service by April 1, 1924, or soon thereafter.

A study of load conditions early in 1923 disclosed the fact that the entire output of the Tugalo plant would be required by the load demands of the system immediately on completion, and, therefore, construction was begun in the early summer of 1923 on two more plants in the Tallulah and Tugalo chain, namely, the Terrora and Yonah Developments.

The Terrora Development lies between the Mathis and Tallulah Reservoirs, and consists of a power-house with 20 000 kw. in generating equipment, located at the head of Tallulah Lake (Fig. 2). A surge tank will be built on the hillside, directly above the power-house, and a tunnel about 5 600 ft. long will extend from the surge tank to the intake structure on the shore of the Mathis Lake, presenting a development similar to the original Tallulah Falls plant, except that the total head is less. The gross head on this plant will be 190 ft., 90 ft. being formed by the Mathis Dam and 100 ft. by the fall in the river between the dam site and the power-house. The generating capacity will consist of two 10 000-kw. generators of vertical type, direct-connected to 15 000-h.p. water-wheels which will be supplied with water from the surge tank, through two steel penstocks 9 ft. in diameter and about 900 ft. long. This plant will have an annual output of approximately 60 000 000 kw-hr. in years of normal water flow, and will utilize the storage waters behind the Mathis and Burton Dams during the dry seasons.

The Yonah Development will consist of a cyclopean dam on the Tugalo River about three miles below the Tallulah plant, and will back water to the Tugalo tail-race. The dam will be 75 ft. high, about 900 ft. long, and will contain approximately 115 000 cu. yd. of masonry (Fig. 2).

The generating equipment will consist of three 75 000-kw., vertical type machines, direct-connected to 12 500-h.p. water-wheels, operating under a gross head of 75 ft. The plant will utilize all the water passing the Tugalo plant from the storage reservoirs on the Tallulah and the unregulated flow of the Chattooga, and is rated at about 70 000 000 kw-hr. per annum.

In addition to the developments described as now existing, or in course of construction, a smaller development will be started in the latter part of 1924, known as the Nacoochee plant, to utilize the 60-ft. fall on the Tallulah River between the Burton Dam and the back-water from the Mathis Dam, consisting of a 60-ft. dam and a power-house containing 5 000 kw. in a single unit. As soon as this plant is under way, a power-house with 6 000 kw. in generating equipment will be built below the Burton Dam, provision having already been made in the dam structure for the intake and penstock installation. These plants are comparatively small, but will increase the total output of the system by approximately 50 000 000 kw-hr.

These plans provide for complete development of Tallulah and Tugalo Rivers throughout a river distance of 38 miles, where the total fall is 1 199 ft., practically every foot of which will be utilized in power production. The total installed capacity of the plants will be 178 000 kw., and the estimated annual output, during a year of normal water flow, will be 531 000 000 kw-hr., based on a net run-off of approximately 40 in. out of a total rainfall of 65 in. as the average over the entire water-shed. Attention is called to the fact that the water stored in the Burton and Mathis Reservoirs is equivalent to 125 000 000 kw-hr. at the water-power plants below the reservoirs, which is approximately 24% of the total annual output of the Tallulah and Tugalo System. These figures do not include the effect of storage reservoirs which some day will be constructed on the Chattooga River, above the Tugalo and Yonah plants; nor do they include the effect of reservoirs which will be constructed in the future on the Chattooga River above the Tugalo and Yonah plants.

Coincident with the hydro-electric developments of the past few years, extensions have been made to the transmission system by the construction of a single circuit, 110 000-volt line from the Terrora plant to a switching station near Tallulah Falls, with a double circuit line of the same voltage from this station to the Tugalo plant, and a single circuit, 110 000-volt, 4/0 line from the switching station at Tallulah through Toccoa, Commerce, Winder, and Norcross, to Marietta, Ga., 110 miles long. New transformer stations have been built on the 110 000-volt system at Cornelia, Toccoa, Commerce, Winder, Aragon, Norcross, and East Point, and the capacity of many stations previously built has been doubled or materially increased. A new 110 000-volt line is now under construction from Lindale through Summerville, LaFayette, and Chickamauga, to Chattanooga, which will be used to supply new transformer stations at the three first-named points, and to establish another connection with the Tennessee System.

Table 1 gives the figures just quoted with respect to water-power plants on Tallulah and Tugalo Rivers, and the capacities of the various 110 000-volt

TABLE 1.—Data Relative to the Tallulah and Tugalo River Development.

Burton Development:	
Height of dam, in feet	116
Drainage area above dam, in square miles	136
Area of reservoir, in acres	2 775
Capacity of reservoir, in cubic feet	5 280 000 000
Capacity of reservoir, in equivalent kilowatt-hours:	
At Nacoochee Power-House	5 540 000
At Terrora Power-House	17 300 000
At Tallulah Falls Power-House	55 000 000
At Tugalo Power-House	13 850 000
At Yonah Power-House	8 600 000
Total	100 290 000
Generating capacity, in kilowatts	6 000
Water-wheel capacity, in horse-power	9 000
Head, in feet	84—116
Average annual output, in kilowatt-hours	32 200 000
Average annual output, in horse-power-hours	48 300 000
Nacoochee Development (Seed):	
Height of dam, in feet	60
Drainage area above dam, in square miles	144
Area of reservoir, in acres	560
Available capacity of reservoir, in cubic feet	350 000 000
Generating capacity, in kilowatts	5 000
Water-wheel capacity, in horse-power	7 500
Head, in feet	60
Average annual output, in kilowatt-hours	20 000 000
Average annual output, in horse-power-hours	30 000 000
Terrora Development (Mathis-Tallulah):	
Height of dam, in feet	90
Drainage area above dam, in square miles	153
Area of reservoir, in acres	834
Capacity of reservoir, in cubic feet	1 369 000 000
Capacity of reservoir, in equivalent kilowatt-hours:	
At Terrora Power-House	2 210 000
At Tallulah Falls Power-House	15 000 000
At Tugalo Power-House	3 940 000
At Yonah Power-House	2 450 000
Total	23 600 000
Generating capacity, in kilowatts	20 000
Water-wheel capacity, in horse-power	30 000
Head, in feet	190
Average annual output, in kilowatt-hours	63 800 000
Average annual output, in horse-power-hours	95 700 000
Tallulah Falls Development:	
Height of dam, in feet	116
Drainage area above dam, in square miles	190
Area of reservoir, in acres	63
Available capacity of reservoir, in cubic feet	63 000 000
Generating capacity, in kilowatts	72 000
Water-wheel capacity, in horse-power	108 000
Head, in feet	608
Average annual output, in kilowatt-hours	200 000 000
Average annual output, in horse-power-hours	300 000 000
Tugalo Development:	
Height of dam, in feet	140
Drainage area above dam, in square miles	481
Area of reservoir, in acres	557
Available capacity of reservoir, in cubic feet	500 000 000
Generating capacity, in kilowatts	50 000
Water-wheel capacity, in horse-power	88 000
Head, in feet	150
Average annual output, in kilowatt-hours	140 000 000
Average annual output, in horse-power-hours	210 000 000
Yonah Development (Ralston):	
Height of dam, in feet	75
Drainage area above dam, in square miles	490
Area of reservoir, in acres	425
Available capacity of reservoir, in cubic feet	350 000 000
Generating capacity, in kilowatts	25 000
Water-wheel capacity, in horse-power	37 500
Head, in feet	75
Average annual output, in kilowatt-hours	75 000 000
Average annual output, in horse-power-hours	112 500 000
Summary:	
Total storage capacity, in cubic feet	7 912 000 000
Total head, in feet	1 199
Total generating capacity, in kilowatts	178 000
Total water-wheel capacity, in horse-power	280 000
Total average annual output, in kilowatt-hours	531 000 000
Total average annual output, in horse-power-hours	796 500 000

transformer stations now in service or under construction, as illustrated on Fig. 1, which shows the Georgia System and its interconnections.

The total generating capacity of the Georgia Railway and Power Company's System, on the completion of the plants now under construction, and those immediately proposed, will be 217 400 kv-a., with an annual output of approximately 600 000 000 kw-hr., all of which will be in service within the next two or three years. To these figures should be added 189 000 kv-a. as the combined generating capacity of the Central Georgia and Columbus Power Company Systems, serving the central part of the State of Georgia, and directly connected with the Georgia System, giving 406 400 kv-a. as the total generating capacity which will be in service within the next few years on the interconnected transmission systems of the State. This does not include the small power developments or isolated steam plants which are not connected to the high-voltage transmission lines.

As previously intimated, the Georgia System is the central system in the Southeastern network, being connected on the west with the Alabama System, on the north with the Tennessee System, and on the east with the Southern Power System, which, in turn, is connected with the Carolina Power and Light Company, operating in the territory northeast of the Southern Power System. The growth of the industry in the adjoining States has been equal to or greater than the development described.

In North Carolina and South Carolina, there will be a generating capacity amounting to 606 550 kv-a., on the completion of the plants listed in Table 2 under the Southern Power Company and the Carolina Power and Light Company. The Alabama System will have a total capacity of 571 800 kv-a. (not including the Muscle Shoals hydro-plant), and the Tennessee System, 196 405 kv-a., making the total combined generating capacity of the Southeastern System, on the completion of plants now under construction, or immediately proposed, 1 781 155 kv-a., with an estimated annual output not less than 5 000 000 000 kw-hr. The total output of the combined systems during 1923 is given in Table 3 as 2 979 543 600 kw-hr., which was 20% greater than the corresponding figure for the preceding year.

The territory within service distance of these various companies is approximately one-half the combined area of the States of North and South Carolina, Georgia, Alabama, and Tennessee, amounting to 120 000 sq. miles, with a total population of more than 6 000 000. It is thickly traversed with high-voltage transmission lines, thoroughly interconnected, and spotted every few miles with high-voltage transformer stations acting as supply points for the lower voltage distributing feeders radiating therefrom. Table 4 shows the approximate miles of high-voltage transmission circuits on the various systems, exclusive of all distributing lines operating at 44 000 volts, 38 000 volts, or less, from which it will be noted that the total actual line mileage of these high-voltage lines on the interconnected systems is 3 112, over which double circuits are strung in many cases, or provision is made for installation of a second circuit at a later date.

TABLE 2.—Showing the Generating Capacity of the Various Companies Comprising the Southeastern Power System.

North Carolina and South Carolina.

Company.	Type of plant.	Existing capacity, in kilovolt-amperes.	Capacity under construction, in kilovolt-amperes.
Southern Power Company:			
Bridgewater	Hydro	25 000	.....
Rhodiss	"	37 500	.....
Lookout	"	22 800	.....
Mountain Island	"	75 000	.....
Catawba	"	6 600	.....
99 Islands	"	22 500	.....
Fishing Creek	"	37 500	.....
Great Falls	"	30 000	.....
Rocky Creek	"	30 000	.....
Dearborn	"	56 250	.....
Wateree	"	70 000	.....
Total	.....	413 150	.....
Mt. Holly	Steam	45 500	.....
Tiger	"	37 500	.....
Greenville	"	8 000	.....
Greensboro	"	8 000	.....
Eno	"	31 250	.....
Total	.....	130 250	.....
Total hydro and steam power	.....	543 350	.....
Carolina Power and Light Company:			
Buckhorn	Hydro	2 900	.....
Blewetts	"	24 000	.....
Raleigh	Steam	3 700	.....
Cape Fear	"	15 000	15 000
Florence	"	1 500	.....
Total steam power	.....	21 250	15 000
Total hydro power	.....	26 900	.....
Total hydro and steam power	.....	48 150	15 000

Note.—Some of the plants listed in Table 2 under Southern Power Company plants as existing are under construction. The table does not include all the proposed plants of the various companies, but only those on which construction has actually started.

TABLE 2.—(*Continued*).

Company.	Type of plant.	Existing capacity, in kilovolt-amperes.	Capacity under construction, in kilovolt-amperes.
TENNESSEE.			
Tennessee Power Company:			
Hales Bar	Hydro	43 430	.....
Ocoee No. 1	"	21 000	.....
Ocoee No. 2	"	18 750	.....
Rock Island	"	9 375	15 000
Total hydro power	.....	92 555	15 000
Parksville	Steam	13 000	.....
Nashville	"	29 600	.....
Knoxville	"	6 250	.....
Hales Bar	"	000	40 000
Total steam power	.....	48 850	40 000
Total hydro and steam power	.....	141 405	55 000
GEORGIA.			
Georgia Railway and Power Company:			
Tallulah Falls	Hydro	72 000	.....
Tugalo	"	50 000	.....
Terrora	"	.....	20 000
Yonah	"	.....	25 000
Burton	"	.....	6 000
Nacoochee	"	.....	5 000
Morgan Falls	"	15 000	.....
Dunlap	"	2 200	.....
Chestatee	"	1 000	.....
Gregg Shoals	"	1 200	.....
Atlanta steam plants	.....	20 000	.....
Total hydro power	.....	141 400	56 000
Total steam power	.....	20 000	.....
Total hydro and steam power	.....	161 400	56 000
Central Georgia Power Company:			
Lloyd Shoals	Hydro	12 000	.....
Macon Plants	Steam	3 000	15 000
Total hydro and steam power	.....	15 000	15 000
Columbus Power Company:			
North Highlands	Hydro	6 900	53 100
Goat Rock	"	16 000	14 000
Bartletts Ferry	"	.....	60 000
Columbus Plant	Steam	9 000	.....
Total hydro power	.....	22 900	127 100
Total steam power	.....	9 000	.....
Total hydro and steam power	.....	31 900	127 100

TABLE 2.—(*Continued*).

ALABAMA.

Company.	Type of plant.	Existing capacity, in kilovolt-amperes.	Capacity under construction, in kilovolt-amperes.
Alabama Power Company:			
Lock 12	Hydro	81 000	.....
Mitchell	"	60 000	.....
Jackson Shoals	"	2 000	.....
Upper Tallassee	"	.....	44 000
Lower Tallassee	"	.....	62 500
Cherokee	"	.....	150 000
Total hydro power	.....	143 000	256 500
Warrior River	Steam	58 300	25 000
Gadsden	"	12 500	.....
Montgomery	"	6 000	.....
Sheffield	"	70 500	.....
Total steam power	.....	147 300	25 000
Total hydro and steam power	.....	290 300	281 500

SUMMARY.

Company.	EXISTING CAPACITY, OR CAPACITY UNDER CONSTRUCTION, IN KILOVOLT-AMPERES.		
	Hydro plants.	Steam plants.	Total.
Southern Power Company	413 150	130 250	543 400
Carolina Power and Light Company	26 900	36 250	63 150
Tennessee Power Company	107 555	88 850	196 405
Georgia Railway and Power Company	197 400	20 000	217 400
Central Georgia Power Company	12 000	18 000	30 000
Columbus Power Company	150 000	9 000	159 000
Alabama Power Company	399 500	172 300	517 800
Grand total, Southeastern System	1 406 505	474 650	1 781 155

Thus far, this paper has described the general interconnected power systems of the Southeast, and of the Georgia Railway and Power Company, in particular.

With regard to the operation of these interconnected power systems, little, if anything, of great interest can be said. In the abstract contemplation of super-power systems, much speculation and calculation is usually entered into in order to determine the operating characteristics and the physical operation, based on hypothetical systems and conditions. As a matter of fact, no unusual operating difficulties have been encountered. The system operates as a whole as simply as each system by itself, and the synchronizing and transference of power over these interconnections are carried out with the same regularity and with the same operating characteristics that are found in each of the

systems individually. It is not even necessary to pay attention to the problem of voltage regulation, as each system, when interconnected, continues its regular operation without regard to the other systems. When two systems with somewhat varying voltages are connected, the voltage equalizes somewhere along the line of the interconnection, and the potential conditions on the separate systems are affected little, if any. There has been no trouble in staying in parallel, except during periods of light load, usually in the early morning hours, and if at that time any great amount of power is being transferred from one system to another, no trouble has been experienced.

TABLE 3.—Total Power, in Kilowatt-Hours, Absorbed by Systems of the Southeastern Power Companies.

Company.	1922.	1923.
Southern Power Company	910 488 432	1 001 026 819
Carolina Power and Light Company	160 584 432	198 452 248
Tennessee Power Company	351 436 289	460 097 505
Alabama Power Company	607 133 242	763 260 963
Georgia Railway and Power Company	286 049 585	347 818 196
Columbus Power Company	113 575 334	132 627 319
Central Georgia Power Company	65 021 460	76 261 050
Total	2 491 288 724	2 979 543 600

TABLE 4.—High-Voltage Transmission Lines of the Southeastern Power Companies.*
(66 000 Volts; 100 000 Volts; and 110 000 Volts.)

Company.	Miles of line.
Southern Power Company	893.60
Carolina Power and Light Company	477.00
Tennessee Power Company	551.00
Alabama Power Company	534.28
Georgia Railway and Power Company	450.40
Columbus Power Company	71.10
Central Georgia Power Company	134.90
Grand total	3 112.23

* The distances represent length of lines, some carrying one or more circuits, and do not include any 44 000-volt, 38 000-volt, or lower voltage distributing circuits.

Sections of the territory have been saved from serious interruptions of power service several times during the period these companies have been so thoroughly interconnected. There have been times when many thousands of employees of cotton mills and other industries have been kept at work by a supply of power originating many hundreds of miles away. It is this insurance against shut-downs, when one company can call on the others for assistance, that the greatest benefit of the interconnection is derived, notwithstanding there is a daily interchange of power throughout the territory which serves to conserve the water resources to a large extent. A word in passing

should be added respecting the lease of the U. S. Government steam plant at Sheffield, Ala. Several of the power companies in the Southeast associated themselves in leasing this steam plant in order that it might be operated during the fall, or during dry weather, to supplement the water-power plants during the period immediately following the World War, when the hydraulic construction had not kept pace with the load. This plant has been operated each year by the Alabama Power Company and power transmitted or relayed to the other companies in the Association up to the capacity of the lines connecting the Sheffield plant with the Alabama System. This is approximately 45 000 kw., and the steam plant has been operated at this load during the fall months for the past three years, with every indication that it will also be operated during the fall of 1924 to the same extent. Power in this steam plant has been relayed, not actually transmitted, from Sheffield, to Raleigh, N. C., a distance of nearly 900 miles. This relaying of power is one of the chief advantages of an interconnected system. Although the power from Sheffield was not actually used in Raleigh, the effect of the interconnected system made it possible to use an equivalent amount of power in Raleigh, it having been relayed from one system to the other and used at the point of greatest advantage, involving the least distance of transmission. During this period, every system in the Southeast has been operated in parallel with every generating station on that system in service.

To an engineer familiar with the enormous investment required in the development of water-power construction of steam stations, and the erection of transmission lines operating at high voltage, the figures quoted give a graphic picture of the enormous development which has been going on during the past ten years in the industrial life of the Southeastern States, and that contemplated in the coming decade by those responsible for its power requirements. It would seem to the writer that the general super-power system must come after the various groups of States are interconnected the power companies of which have an immediate mutual interest, and when their development has grown to a point where the interconnection over wide territory will prove economical from the interchange and insurance of load standpoint, to the extent that will warrant the enormous expenditure involved in lines of large capacity and super-voltage.

DISCUSSION

JOSEPH HYDE PRATT,* M. AM. SOC. C. E.—The speaker agrees with the author that the super-power survey is more a process of evolution and growth than of investigation and work. Hydro-electric development in the Southeastern States is of such a character and is being conducted in such a manner that in the near future there will be an interconnecting and interchangeable system of power lines extending into all parts of the seven Southern States beginning with the southern part of West Virginia, and extending across Virginia, North Carolina, Tennessee, South Carolina, and Georgia, to Alabama.

The author has shown clearly and comprehensively how successfully the Alabama Power Company, the Georgia Railway and Power Company, the Southern Power Company, and the Carolina Power and Light Company have operated in relaying power from the steam plant at Sheffield, Ala., to Raleigh, N. C., a distance of about 900 miles. He has also called attention to the co-operation of these several companies in leasing the United States Government plant at Sheffield, Ala., such co-operation or pooling of interests being the beginning of a wonderful power organization that will insure a more efficient relaying of power in the Southeastern States. This is a wonderful advantage to the producer and consumer. It has meant that the co-operating companies have been saved from serious interruptions of power service, and that they have been able to supply power at all times to their customers, which is a great advantage to both companies and consumers.

There is a still greater advantage in this co-operation, that has accrued to these Southern States. Thousands of employees in the cotton-mills and other industrial plants have been kept at work by virtue of the power supply furnished to their plants from a distance. This insurance against shut-downs is one of the greatest benefits derived from interconnection, and one that is of great advantage to the State; it should be particularly impressed on the consumer and on the people of the several States.

As is well known, there has been a great deal of controversy between the producers and consumers, particularly over the question of rates. In some States this has reached the point where a considerable number of the people have been made to feel that the power companies have obtained a monopoly and are trying to impose upon the industries of the State, or to get control of all the power, and thus have the public at their mercy. Considering what these power companies are really doing—how they are co-operating with each other to make more effective and efficient the distribution of power throughout the State, and are trying to get this message regarding the power companies to the people, as to what it means to the State as a co-operative effort—it ought soon to be possible to create a better understanding between consumer and producer, between the people who are dependent on work and supplies, on the power that is developed by these power companies, and on the companies themselves. In the season of 1921-22, when the relaying of power from the Sheffield plant to Raleigh had made it possible for the Carolina Power and Light Com-

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pany to distribute power to its consumers, it meant the keeping open of the mills and factories in that section of North Carolina, and brought home to the public the fact that the power companies were trying to be of some service to the people, and not simply making an income out of them.

A wonderful organization, such as that referred to, is practically in operation now. It is not a corporation, nor is it an organized association of any sort, but it has been a collaboration of these power companies, all experiencing the need at certain times during the year for power obtainable from another company. Probably in the end there will exist in the Southeast what has been called and asked for throughout the country—a super-power system, rather than an interconnection. There is no direct interconnection from Alabama to Raleigh on account of the different voltage of the transmission lines, but it is possible to relay power from Sheffield to Raleigh, and, in a short time, there will be a similar interconnection and relaying system from North Carolina into Virginia and Southwest West Virginia. Then, these seven Southern States will be interconnected and be able to relay power from one to the other.

More serious consideration should be given to the location of auxiliary steam plants, and particularly to the location of co-operative auxiliary steam plants. Consideration should also be given to the location of these steam plants at or near coal mines. In the past two years, there has been developed in Central North Carolina a deposit of approximately 67 000 000 tons of good coking coal, comprising the extreme eastern outlying coal-field. It is recommended to construct at that coal-field a coking plant to utilize the gas in an auxiliary steam plant and the coke for a domestic fuel, this method being deemed more efficient and economical than a coal-operated, auxiliary steam plant.

The work of the Georgia Railway and Power Company in the development of what is called the Tallulah System is excellent, because it considers the river development as a whole. There has been a similar investigation on two or three rivers in North Carolina during the past few years by Thorndike Saville, Assoc. M. Am. Soc. C. E., Hydraulic Engineer of the North Carolina Geological and Economic Survey, with the object of having these rivers considered and developed as a unit.

William P. Hammond,* Esq.—The speaker has been particularly impressed with the author's statement that the system in the Southeastern States is an interconnected system, rather than a super-power system. The term, "super-power," conveys the idea of a high-voltage transmission system which has been constructed and is operated for the sole purpose of transferring large blocks of power from one locality to another, or from one system to another, first in one direction and then in the opposite direction, to which no customers are connected, except the large generating and transmission companies operating in the various territories traversed. If this definition is correct, the system under discussion is in no sense a super-power sytem, but rather a group of small transmission and distribution systems which have been thoroughly inter-

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connected. Practically all the transmission lines on this system were built for the sole purpose of transmitting power from some particular plant to some particular load. In addition, they may now supply many other smaller transformer stations en route; the use of such lines as now exist for transferring power from one company to another has been a distinctly secondary consideration.

The system under discussion is the result of a perfectly logical and evolutionary development which has taken place step by step to meet the growing demands for power in this territory and is not in any sense the result of a study and a predetermined layout for the territory as a whole as would be necessary in the construction of a true super-power system. The development of water power on a commercial scale, as pointed out in the paper, began almost simultaneously in the few years prior to 1912, in the States of North Carolina and South Carolina, Alabama, Tennessee, and Georgia. Plants were built in each of these States, and transmission lines constructed between these plants and the nearest load centers. As soon as these first lines were built, other loads were developed near-by and new extensions were made to supply these new loads, the revenues to be received from the new connections, in each case, justifying the construction expense involved. As each system expands in this perfectly logical manner, it soon happens that they are approaching or touching each other at various points, so that the interconnection of the several systems then involves the construction of relatively short lines, and the expense of this interconnection can be justified from an economical standpoint, although the interchange of power over the new connection is not extensive. No matter how important a line may be from a technical or engineering viewpoint, it is exceedingly difficult to secure the necessary appropriation for its construction unless the returns will warrant the expenditures.

The power companies in these five Southeastern States have developed water power and constructed steam stations of the approximate capacity necessary to satisfy the load on their respective systems. The interconnections have been utilized primarily in passing power from one system to another, at certain periods of the year when a water shortage exists on one system that may not exist to the same extent on another, or when steam power could be produced on one system at a lower rate than on another, or at other times in emergency situations. This means that large blocks of power have not been passed over the interconnections continuously; the greater part has passed in one direction, namely, from the West to the East on account of the large reserve steam capacity on the Alabama System, but smaller amounts have also been transferred at other periods in the opposite direction. Thus, the figures for any particular year representing net power delivered in one direction over any of the interconnections appear small. The value of the connection in such emergency situations cannot be set down in dollars and cents, based on the actual power transferred, but the net figure, referred to, is the only record at the end of a year as to the extent to which the connection has been utilized. These figures, as intimated, are not sufficiently large to justify large expenditures for interconnecting lines. On account of these very considerations, it is

impossible to justify the construction of a real super-power system as herein defined in the Southeastern territory at the present time. As stated by Mr. Adsit, the system which has been described as now existing in this Southeastern territory is simply a group of interconnecting transmission systems, and not a real super-power system.

On account of the large territory covered by these connecting systems, and the rapidity with which the entire development has taken place, many descriptive articles have appeared in the technical press, and it has frequently been held up as a model super-power system. It attracted particular attention during the World War in connection with the surveys which were made by the War Department on the power resources of the different sections of the country, and many of the articles which have appeared have been written by Government engineers, or by men formerly employed in connection with these surveys. It seems to be the general impression that the development has come about largely through the recommendations made by Government engineers in reports on the power resources of this section. This is not the case, as all the large systems comprising the interconnected system of the Southeast were joined before the Government undertook these studies, and only a few additional connections have been established since the initial report was submitted in March, 1919. These connections would have been established without the Government surveys. The engineers representing the various power companies do not claim to have any monopoly on information pertaining to the possibilities and advantages of interconnections of this nature. Many of the recommendations in the Government report are well made, but the real credit for the progress thus far must be given the engineers of the power companies themselves, and not to the Government, it being true that, in the early stages of the development, the Government actually did more to prevent these connections from being made than to assist in the work. It is the general feeling among the engineers of the power companies that the part taken by the Government in the development of water power and interconnection of transmission systems, through reports and recommendations, has hindered rather than promoted plans.

There are some few disadvantages of interconnection, but these are of only minor importance. There is, however, in many of the articles on this subject, an apparent tendency to exaggerate the benefits which result from such interconnection and to picture the possibility of a real super-power system far out of proportion to what can be economically justified in the immediate future. Some writers foresee real super-power systems, extending almost entirely across the United States, into which power may be delivered by any company that is connected and has a surplus above its own requirements and from which any other companies may take power when their load requirements are excessive.

It is claimed that large hydro-electric developments could then be prosecuted continuously and tied into the super-power system in logical order, based on the cost of any development and the value of its output to the territory as a whole rather than to some particular company. Large plants would be

loaded promptly, thus avoiding the loss now incurred by the separate companies during the interval between the completion of the plant and the growth of its load. Steam capacity to serve during low-water periods could likewise be built in large units having their location selected with proper regard to the fuel supply and water conditions, rather than to its proximity to some particular system. Under this plan, all the hydro-electric power plants might be located in one State on the system of one company, and all the steam plants in another State on the system of another company; both companies, being connected to the super-power system, would derive the combined benefits.

In the light of developments during the past ten years, it would be foolish to state that such a super-power system will never exist, as present tendencies are undoubtedly in this direction. It must be self-evident that such a system can only exist and be operated to maximum advantage when the generating, transmission, and distribution systems of the entire territory covered by the super-power lines have been brought under common control or management. Past experience does not indicate that the Federal Government or the people particularly desire to take this course.

Dana M. Wood,* M. Am. Soc. C. E.—There has been much discussion recently concerning the advantages obtainable from the interconnection of power systems. Several large systems in different parts of the country already have agreements for the interchange of current; and operating economies have resulted. There is, however, need for some intermediate controlling agency in order that still further economies may be effected.

Interconnection problems are developing a new type of calling, although not yet so recognized. The railroad companies found it useful to create the official position of "Train Dispatcher"; soon interconnected power systems may require the filling of a comparable position to be known perhaps as "Load Dispatcher".

To obtain the highest efficiency in generating and transmitting power in the combined territory of several power companies this Load Dispatcher should be independent of each individual company, but responsible to all. His duty will be to familiarize himself with the fundamentals of rainfall and run-off throughout the territory in which the generating plants are located and to compile all useful operating facts and data pertaining to each plant and company. He will be called on to predict, as accurately as present knowledge allows, the probable supply and demand for the near future. He must know the cost of power from the several sources available. As a result of such studies, he must control the output of the various plants for the benefit of the combined territory served. He must anticipate shortages or surplus of power at individual plants, and attempt to prevent their occurrence.

This ideal arrangement is only doing on a larger scale what each company having several generating plants is at present doing, or ought to be doing, for itself. It cannot come to pass, however, until the various companies are willing to delegate large powers to the Load Dispatcher and his organization. Perhaps, in the final analysis, it may be necessary to bring about some financial

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as well as physical interconnection between companies, that will tend to eliminate the obstacle of selfish company interests. That such an idea is practicable has already been demonstrated by the late Walter H. Sawyer, M. Am. Soc. C. E., who succeeded in inducing the various concerns along the Androscoggin River, in Maine, to pool their interests to the extent of developing storage and manipulating run-off for their maximum benefit as a group.

GEORGE A. ORROK,* M. AM. SOC. C. E.—It is well known that the low-water flow of the Tennessee River between Hales Bar and Muscle Shoals varies from 3 500 to 5 000 sec-ft. The Government requires that the low-water flow fixed at 5 000 sec-ft. shall be maintained whether or not power is being generated. This would mean at Hales Bar on a 39-ft. head the production of 16 800 kw. and at Muscle Shoals probably 48 000 to 50 000 kw. One hears 200 000 kw. spoken of very glibly, and even 1 000 000 h.p., but these figures do not quite square with the facts. In discussing a water-power plant, even if it includes storage in the head-waters, well known flow records should not be disregarded; and the flow records on the Tennessee River have been kept rather accurately for many years.

Those who have had to do with the development of steam-power plants are also greatly interested in the building of steam plants at coal mines. Unfortunately, for every ton of coal burned in the steam-power station, from 600 to 1 000 tons of water are necessary. Almost invariably this condensing water has to be pumped; there are only one or two exceptions in the United States. In the North 600 tons of water are required per ton of coal, and in the South it would be more like 1 000 tons.

It is better to haul coal on a railroad than to pump water any great distance or attempt to use cooling towers as they do in England or Germany. Recently, the speaker visited a station, built in rather close proximity to the coal mine, that has been forced to use the sewage discharge of a large city for its supply of condensing water. In order to cool this discharge they have to use towers, so that the station is surrounded by a forest of cooling towers, thirty-seven in number, 35 ft. wide by 100 ft. long. They have rain from the cooling towers every afternoon. There are power-station locations at the coal mine, with a river in close proximity, an ideal location.

The speaker has followed this subject of interconnection for a great many years. It is only of use where diversity exists. There is a wonderful interconnected system in the City of New York, which is useless because there is no diversity. The peaks of demand come within 3 or 4 min. of each other. Whenever one system wants power, the other system also wants it, and there is no chance for interchange except in case of a breakdown. It is true they do interchange power in the New York District to the extent of probably 70 000 or 80 000 kw. of a total load of 1 600 000 kw.

In other cases, there is diversity of load and diversity of water supply, so that interconnection can be of great value. This has been exemplified in the South, in the Pittsburgh District, in the vicinity of New York outside of the Metropolitan District, in the West around Chicago, Ill., on the Pacific slope,

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in Italy, in Switzerland, and, the speaker understands, in France and Germany. The interconnection of the electrical transmission lines in Northern Italy is of use because of the diversity of the water power, some of the rivers obtaining their water from the melting of the Alpine snows, others from rain falling on the Apennines.

The interconnecting systems in Switzerland are not quite so diversified; they are connected in three places to Germany on the north, two to France on the west, and four to Italy on the south, but to date little power has been transferred. The interior connections are particularly useless in that respect. It is hoped, however, that in the course of a few years more sources of power will be developed and more diversity will be secured. In their interconnected systems along the Belgian line, and also in the south of France, French engineers are attempting to tie in with the distribution system of Paris, and by those connections to secure both diversity of load and supply.

As to interconnection in the Northern United States, it is not generally known that Manlius, N. Y., and Portsmouth, N. H., are already connected. The Adirondack Company, in connection with the Syracuse Company, is building a tie line of 12 miles between Manlius and Syracuse. This will enable the Company to transmit power from Portsmouth to Windsor, Ont., Canada. Power is now being transmitted or interchanged from the Adirondack System to the New England Power System, which includes a stretch practically 500 miles in length.

E. A. Yates,* Assoc. M. Am. Soc. C. E.—The seven major power companies of the Southeast are the Carolina Power and Light Company in the eastern portions of North Carolina and South Carolina; the Southern Power Company in the middle portions of North Carolina and South Carolina; the Georgia Railway and Power Company in Central and North Georgia; the Central Georgia Power Company at Macon, Ga.; the Columbus Power Company at Columbus, Ga.; the Alabama Power Company in Alabama; and the Tennessee Power Company in Tennessee. All these companies have interconnection and realize its value. Largely by reason of the efforts of the Georgia Railway and Power Company, the present interconnecting lines have been built. This Company being geographically at the center, or so-called "hub", it has been logical that the interconnecting lines should develop radially from it. Considering that the interchange of power between these companies was approximately 85 000 000 kw-hr. in 1922, and nearly 240 000 000 kw-hr. in 1923, and that some of the lines were operated in 1923 largely in excess of their rated capacity, it would seem that more interconnections are necessary. They are needed along the outside, that is, from outlying companies. The Alabama Power Company and the Tennessee Power Company are now contemplating a high-tension connection between Alabama and Tennessee, and a new connection will be made in the near future through the Tennessee Valley, connecting the Tennessee power lines at Knoxville, Tenn., and the lines of the Southern Power Company in North Carolina. This connection will provide a second trunk line from Alabama to North Carolina.

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The advantages of interconnection are obvious to any student of power distribution. The increase in reliability of service, the guard against breakdown, the absorption of excess capacity from the plant by all the companies during the period of loading, and the greater utilization of water, are all arguments favoring interconnection and interchange of power.

Co-ordination and co-operation between the large companies emphasizes the principle that power facilities should be created with a view to the aggregate and not the individual demands, reduction of costs, and reliability of service. This does not mean encroachment of one power company on the territory of a neighbor, nor that one company supplies another with average load energy, but it does mean closer co-operation and use of excess capacity. The joint-load principle is recognized as desirable, and, in the future, a closer co-operation will be carried even to the investment of capital of connecting companies to provide for power-producing facilities. There are a limited number of streams and rivers susceptible to development, and a limited quantity of power capable of development; nothing can justify a lack of co-ordinate development and operation of these sites so that, as nearly as possible, they may provide 100% use. The steam plants, the storage water plants, and the run-of-river plants, of all these companies should be operated so as to utilize the greatest quantity of water and burn the least coal.

These seven companies generated approximately 3 000 000 000 kw-hr. of energy during 1923 (an increase of 20% over 1922), and each company is now engaged in the construction of one or more generating plants, both steam and hydro-electric, to keep abreast of local market demands. It is reasonable to believe that this rapidly increasing demand for energy in the South is to continue. Climate, labor, and transportation have all been large factors in the development of the South; reliable and reasonably priced power has been one of the greatest factors.

Interwoven with the network of transmission lines and generating plants of these companies is the power situation at Muscle Shoals. Located, as it is, in the northwestern part of Alabama, on the Tennessee River, near the State line, this great source of water power, an unusually large proportion being secondary, is exceptionally well situated for use in a super-power distribution system in the South, extending to the western part of Tennessee and to Mississippi. Only by the operation of the Muscle Shoals plant in co-ordination with the power-producing facilities, both water and coal, of these States can the greatest utilization of these developments be realized. The Muscle Shoals development is a run-of-river plant and, although it will eventually have an installation of approximately 800 000 h.p., the low flow of the Tennessee River reduces the absolute primary power to about 140 000 h.p. In order that this power development may be utilized to the utmost, it is essential that it be supplemented by steam and by the storage water plants of the companies of the Southeast.

The Muscle Shoals section of the Tennessee River covers a fall of approximately 135 ft. The dual purpose of navigation and power will be served by the construction of two dams; the Wilson Dam, known as Dam No. 2, with a

head of 93 ft., is now nearing completion. Construction of Dam No. 3, up stream, with a head of 40 ft., will be started on completion of the present work.

In 1914, the Alabama Power Company acquired all the lands and rights at these dam sites and expended approximately $500 000 in land, borings, surveys, and engineering studies looking toward the development of these two sites for the joint purpose of navigation and power.

On two occasions prior to the World War, agreement was reached by the Power Company and the Board of Engineers of the United States Army, whereby in part the expense of the development would be borne by the United States in the interest of navigation, and in part by the Power Company in the interest of power.

At the beginning of the War, it was decided to establish the great nitrate plants at Muscle Shoals, building Dam No. 2 to supply the power. (Here, also, it is proposed that fertilizer production shall be carried on by the manufacture of nitrogen and other ingredients of fertilizer in a concentrated form.) It is not generally known that the Alabama Power Company gave the Government all its rights to Dam No. 2 at Muscle Shoals, representing in the aggregate approximately $500 000. Further than that, it built an addition to its steam reserve plant at Warrior, 90 miles south, and installed a 30 000-kw. unit in order that the Government might have power at an early date for the construction and operation of the nitrate plants. This extension utilized the intake and discharge culverts, foundation, coal mines, tracks, and lines of Alabama Power Company. The Government recently sold this extension to the Power Company for $3 500 000, which was just and proper, and returned to the Government a large part of its expenditures on this project.

Thus, at the end of the War there was at Muscle Shoals Nitrate Plant No. 1 and Nitrate Plant No. 2, which cost the Government in excess of $80 000 000, together with a 60 000-kw. steam plant costing approximately $12 000 000, and Wilson Dam, partly finished, on which the expenditure of approximately $17 000 000 had been made.

In 1921, in response to a request from the Government, the Alabama Power Company made a further proposal for co-operation and development of Muscle Shoals, and again in 1922, still another proposal to complete this dam at its own expense, under the provision of the Federal Water Power Act, and to provide free 100 000 h.p. for use in the manufacture of fertilizer.

The industrial growth in the South during the past ten years has been largely in cotton mills, steel plants, coal mines, iron mines, and various factories, all of which use power, and in the development of which reasonably priced, dependable power has been a large factor. In 1923 these industries paid on an average approximately 1 cent per kw-hr. for their energy.

EDGAR A. VAN DEUSEN,* ASSOC. M. AM. SOC. C. E. (by letter).—It is inspiring to see a map of the Southeastern States showing a great interconnecting and interstate power system that supplies with certainty and economy the

* Asst. Hydr. and Structural Engr., Central Hudson Gas & Elec. Co., Poughkeepsie, N. Y.

diversified needs of a large and scattered population. The really significant fact, however, is that this achievement is the result of private enterprise—a monument to the American doctrine of co-operative individualism as opposed to the socialistic doctrines of Europe.

It is very improbable that governmental (political) ownership could have developed the "Interconnected Power Systems of the Southeast." It is conceivable, of course, that one State, through the efforts of a man of vision and forceful personality—a Sir Adam Beck—might develop its power systems into an effective combination; but for five adjacent States to do so contemporaneously, and, in addition, to agree to mutual interconnection and energy interchange, would require Utopian conditions. Consider, as examples, the present Colorado River and the St. Lawrence River controversies, in which, despite the earnest efforts of qualified and conscientious engineers, and other patriotic citizens, politics seem to be the dominant factor. Analysis shows that even the famed Ontario Hydro-Electric System, splendidly conceived and executed as it is, the praises of which are forever being sung by proponents of Governmental ownership, is a relatively expensive experiment in which political expediency looms large.*

The interconnection of power systems, as distinct from the super-power idea, is making great strides in America. New England, New York, Pennsylvania, the Middle West, the Northwest, and the Pacific Coast, as well as the South, can point to numerous examples of this, both interstate and intrastate. Maine, by reason of its prohibition of power export, is the only exception. Super-power systems will doubtless evolve from these projects in the course of time.

The author has drawn attention to the comprehensive scheme of power development for the Tallulah River, in which 90% of the annual run-off will be available for power generation. It is gratifying to realize that this, like interconnection, is also a trend of the times. The Catawba River, the Deerfield River, and the Au-Sable River (Michigan), are three other examples, in which a high percentage of the total run-off is transformed into power. There are doubtless other instances, to say nothing of the many projects which have been highly developed on paper, and are merely awaiting improved economic conditions before they take concrete form.

Modern business is striking a new note, "Service"; especially is this true of the public utilities of the country. The old ideas, "the public be damned", "business is business", etc., are going the way of the horse car and other anachronisms; but in spite of this and of the additional fact of "Government regulation" a safeguard for the people which practically precludes the possibility of exploitation there are still those who, honestly or otherwise, urge Government ownership and operation of some or all public utilities. Engineers owe it to themselves and to their country to examine the evidence, to draw the logical conclusions, and to educate their misinformed fellow citizens.

* See National Electric Light Assoc. Report by Mr. W. S. Murray, and the paper entitled "Truth About Electric Rates in Ontario", by Mr. W. M. Carpenter in *Electrical World*, October 25, 1924, p. 887.

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IMHOFF TANKS—REASONS FOR DIFFERENCES IN BEHAVIOR*

By Harrison P. Eddy,† M. Am. Soc. C. E.

With Discussion by Messrs. George T. Hammond, John F. Skinner, John R. Downes, David A. Hartwell, Morris M. Cohn, A. P. Learned, Karl Imhoff, and Harrison P. Eddy.

Introduction

The object of the investigation reported in this paper was to ascertain, in so far as possible from available information, the reasons for the differences in operating results obtained from Imhoff tanks, in order that the requirements for satisfactory results might be better understood. The plants at Schenectady, N. Y., Plainfield, N. J., Fitchburg, Mass., and Rochester, N. Y., have been selected for comparison, for the following reasons: Two of the plants have functioned in a normal manner and two have developed serious difficulties; the operating data are unusually complete and reliable; and all the plants have had close, intelligent supervision.

Theory of Operation of Imhoff Tanks

There are two primary functions of Imhoff tanks: (1) The deposition, in the upper or sedimentation compartment, of suspended solids contained in the sewage; and (2) their conversion into inoffensive, relatively stable matter—a process commonly referred to as digestion—in the lower, or digestion compartment.

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The degree of removal of suspended solids from sewage depends primarily on the length of the detention period, although tank efficiency may vary greatly, according to operating conditions.

It is possible that the more prolonged detention period provided in some sedimentation compartments, resulting in greater removal of solids, has had an indirect influence on the process of converting the solids into stable matter, because of the removal from the sewage of a larger proportion of the finer particles, which are more susceptible to digestion.

The principal difficulties experienced in the operation of Imhoff tanks have been connected directly with the action going on in the digestion compartments. This part of the tank may be subdivided into sludge and scum compartments, separated only by an imaginary horizontal plane passing through the slots. It should be recognized that sludge must not be allowed to rise to within less than 18 in. of the slots, and that scum should not extend downward to within that distance above the slots. For convenience, these parts of the sludge and scum compartments may be called the "neutral zones".

Solids that pass through the slots may be divided into two classes, namely, those of greater specific gravity than the sewage, which settle to the bottom, and those of less specific gravity, which float in the scum compartment. The proportion of solids of the latter class is small. Such solids comprise substances which have been entrained in the mass of heavier solids and thus caused to settle.

The solids which are removed from the sewage by sedimentation are, with the exception of sand and mineral detritus from storm water, and certain industrial wastes, essentially organic, although such organic matter contains mineral ingredients. These organic substances are, or soon become, decomposed owing to the action of bacteria and other living organisms, or their enzymes, present in the sewage and propagated in the digestion chamber.

In time, these organisms decompose the organic matter into gases, soluble substances, and insoluble material. It is true that some kinds of organic matter are more easily and more quickly decomposed than others. The finely divided solids, which are more largely removed in tanks providing long detention periods, probably belong in the former class.

Of the products of decomposition, the gases escape by solution in the liquid and by passage through it into the atmosphere; the soluble substances are dissolved in the liquid in the digestion compartment, from which they escape either by diffusion through the slots or by withdrawal of the sludge (in either case, the liquid is replaced by sewage from the sedimentation compartment); the stable solids remain as sludge and scum in the digestion compartment. It appears, in general, that from 30 to 50% of the weight of solids removed from the sewage escape in solution or as gas, leaving 70 to 50% as solids in the sludge and scum.

The digestion of the organic matter results in the production of a practically inoffensive residue, or sludge, which is generally black, somewhat flocculent, well filled with gas as drawn from the tank under ordinary conditions, and drains and dries readily when spread on porous beds of sand or similar material. Digestion sufficient to produce these character-

istics may be considered complete for practical purposes, although this process may be carried further.

Time is required to complete the conversion of the fresh sewage solids into well digested sludge, after the ripening process has once become established, the length of time being greatly influenced by temperature. If the sludge is kept at a temperature just above freezing, scarcely any digestion will take place; if heated to 70° Fahr., digestion will proceed rapidly.

Theoretically the well digested sludge should lie at the bottom of the accumulation and the fresh sludge at the top, there being a gradation in quality between these two extremes. Therefore, when sludge is withdrawn, it should be taken from the bottom.

In northern climates such as those in the four cities under consideration, there is a great variation from season to season in the temperature of air, sewage, and, most important of all, solids and liquids in the digestion compartment. In summer, when the temperature in the sludge compartment may be from 65° to 70° Fahr., digestion is rapid, as indicated by vigorous evolution of gas, but, in winter, it is greatly retarded. The digestion process follows a seasonal cycle. In summer, the sludge may be in excellent condition, deteriorating during the fall, becoming quite offensive in winter, and gradually improving in the spring. In some cases, sludge withdrawn in winter is of poor quality and quite offensive. Obviously, therefore, the digestion compartment must be of sufficient size to accommodate the solids deposited in the late fall, winter, and early spring, so that it will not be necessary to withdraw partly digested and offensive sludge.

The solids as first deposited form a thin, voluminous sludge. As decomposition progresses, they are disintegrated and, being finer, tend to become compact. This tendency is aided by the weight of solids subsequently deposited. As a result of these conditions, the mass varies in consistency from a thin, watery material at the top to a comparatively thick sludge at the bottom.

Gases generated during the decomposition of the solids gradually increase in volume until they permeate and buoy up masses of sludge which pass into the overlying liquid. If there is opportunity for the gases to escape from the solids in which they are entrained, the solids will return to the sludge mass, because of their relatively high specific gravity. In practice, however, the gases escape with some difficulty, and masses of sludge are carried into and remain in the scum compartments, thus forming a heavy scum of very much greater volume than those floating solids which are actually of lighter specific gravity than the sewage. If the sludge or the scum becomes extremely compact, there is not opportunity for the proper functioning of the organisms, and the digestion process is greatly retarded and may be quite incomplete.

Difficulties in Operation

At some plants much of the gas rising in the vents has produced a light, voluminous foam, due to tenacious films enclosing the gas bubbles. At times, this foam has accumulated in considerable masses, filling the gas vents, overflowing their walls, and spreading out and covering walks and the sewage in the sedimentation compartment.

Excessive accumulation of gas-lifted solids—scum—in the gas vents, has been another serious obstacle to successful operation. In some plants, the scum has risen above the tops of the gas vents and, in others, it has extended downward practically to the elevation of the slots. Under the latter condition, solids have been carried upward through the slots into the sedimentation compartment, from which they escaped with the settled sewage, greatly decreasing the efficiency of sedimentation.

Under some conditions the proportion of solids remaining in the bottom of the tank has been so small that it has been practically impossible to remove any solids in the form of sludge without first driving down the scum by breaking it with paddles or streams of water to liberate the gas. If scum is thus broken and the solids are allowed to settle, they rise again in a comparatively short time.

The reduction of the scum by hand methods or by hose streams, involves a comparatively heavy operating expense. Mechanical agitators are said to have been successfully used for breaking up scum at Newton, Kans., and at Enid, Okla.*

In some plants difficulty has been experienced in obtaining an inoffensive sludge that will dry rapidly on sludge-drying beds. Offensive odors have escaped from the Imhoff tanks at some plants, although generally speaking these tanks are practically free from odors.

Comparative History of Plants

At Schenectady, the Imhoff tanks first operated in January, 1915, have never given entirely satisfactory results and have proved to be very expensive to operate. At times, they have produced great quantities of foam which overflowed the gas-vent walls, the free-board being originally slightly less than 12 in. There has never been much foaming during cold weather. The introduction of lime into the sludge compartment seemed, sometimes, to reduce, and at other times, to increase, the tendency to foam. In all instances, it appeared to modify biological action in the sludge in such a way as to make it offensive.

During the earlier years, the foam was prevented from overflowing the gas-vent walls by playing a hose stream on the scum in the vents. This operation required much labor and at times seemed to increase the foaming. During 1916, the walls of the gas vents were increased about 14 in. in height by means of wooden boxes, which provided a free-board of approximately 26 in. The sludge riser pipes terminate in an open bell about 18 in. below the top of the boxes, and in the center of the gas vents. Since the gas vents were raised, foam has been drawn into the sludge risers, from which it flowed to the sludge drains, when necessary to prevent the foam from overflowing the gas vents. This has resulted in a great reduction in the labor required to prevent foaming. Frequent removal of sludge from the digestion chambers has also been of assistance in preventing foaming.

There has always been a tendency for large quantities of scum to form in the gas vents. Some of this scum has been removed through the sludge

* *Engineering News-Record*, August 23, 1923, p. 301.

riser pipes and, at times, has been comparatively free from odor; at other times, it has been offensive. During 1922, the scum was allowed to accumulate throughout most of the year, in order to determine whether such accumulation would prevent foaming, and if digestion or other actions would finally cause the solids to settle into the sludge compartment. Foaming was practically eliminated by this treatment, but the solids continued to form scum until it extended down practically to the elevation of the slots, after which solids escaped through the slots into the sedimentation compartments in such quantity as greatly to reduce the sedimentation efficiency, to cause unsightly scum on the surface of the sewage in these compartments, and to increase the clogging of the trickling-filter nozzles due to the solids carried out of the tanks. The experiment was finally abandoned, some scum being shoveled out of the gas vents, and some being broken by a self-propelling nozzle attached directly to fire hose. The sludge in the tanks finally was pumped out with the exception of a few hundred gallons left in each hopper for the purpose of seeding the incoming material when the tanks were replaced in service.

After the tanks had been again put into operation, an attempt was made to prevent the solids from remaining indefinitely as an accumulation of scum. As soon as scum began to form in considerable quantity, the gas vents of the tanks, with one exception, were attacked by hose streams to break up the scum and to liberate the gas, after which the solids promptly settled into the sludge compartments. Although further gas generation gradually caused the formation of scum, it was possible to draw sludge from the digestion compartment for one to two days after the hose treatment. At the end of three days, practically all the solids again rose into the scum compartments.

The labor required for liberating the gas from the scum, whether by use of paddles or hose streams, and, in fact, any work required in the gas vents, is exceedingly laborious because of the large number of such vents, 16 per tank, or 144 for the whole plant, which treats about 6 000 000 gal. of sewage daily.

The scum became very dense and compact if allowed to accumulate for a long period, a condition unfavorable for digestion. During 1923, when the scum was broken up frequently throughout the warmer season, digestion appeared to proceed moderately, but not vigorously. The sludge removed was not extremely offensive, although it was not free from objectionable odor. It contained a comparatively small proportion of solids and showed that digestion had not progressed to the necessary degree of completion.

The sludge at Schenectady has always been comparatively thin and has not dried as well as at some other plants. During the summer and early fall, it has usually not been offensive; during the winter and early spring, however, digestion appears to have been reduced to such an extent that the sludge is exceedingly offensive. Although there has been comparatively little odor about the tanks, the slight discoloration of paint on the wooden boxes is evidence that some hydrogen sulphide has been liberated.

At Plainfield the Imhoff tanks, first operated in November, 1916, have been subject to excessive foaming, and a great deal of scum has been removed from the gas vents in order to maintain the plant in efficient and presentable condition.

During the early years of operation, foam overflowed the gas-vent walls and at times covered the sedimentation compartments to a depth of 1 ft., or more. Continuing during at least one winter, it rose in the gas vents, and froze in conical mounds in the form of stalagmites that extended 4 ft. above the gas vents.

Numerous methods to prevent foaming were tried, but none was entirely successful. After about two and one-half years of operation, a Riensch-Wurl screen with ⅛-in. slots was provided for screening the incoming sewage. At the same time, the method of operation was changed from that of using all the tanks at once to only three at a time, in order to increase the velocity of flow through those used, and thus obtain a more uniform distribution of sludge in the several hoppers.

The remaining three tanks were kept detached, in order to allow digestion to proceed undisturbed. The tanks were operated in rotation, the schedule being such that one was removed from service and one placed in action each day, thus allowing each tank to receive sewage for three days, and then to be quiescent for three days. Although foaming has been reduced since these changes were made, it is still much more prevalent than at some plants, and as a preventive measure, it is necessary to remove scum from the gas vents by manual labor. It is said that objectionable odors have been traced to the Imhoff tanks. The sludge has always contained a comparatively small proportion of solids and has not dried as rapidly as would be expected with a normal sludge.

At Fitchburg, the Imhoff tanks, which were first operated in October, 1914, have been free from persistent foaming and excessive scum formation.

During August, 1915, Tank No. 1, which had been in continuous use since the previous October and had had some sludge withdrawn, became very active. Foam rose to a maximum height of 4½ ft. above the sewage level in the central gas vents which are about 5½ ft. high, but did not appear in the side vents. Shortly before this, humus tank sludge had been introduced into this unit. The sewage was shut off from this tank, in order to make repairs, for about two months. After operation was resumed, there was no further trace of foaming throughout the year. Another tank which had been operated in parallel with Tank No. 1, showed no signs of unusual activity.

During June, 1916, foam rose 18 in. in the east row of side vents of Tank No. 1, and overflowed into the sedimentation chamber. Disintegrating the scum with a water-jet had no permanent effect, but the drawing of sludge caused the foaming to cease.

During August, 1917, after a week of very hot weather, there was some foaming in the central chimneys of Tank No. 5. The sludge compartment was found to be filled to the slots. After sludge was withdrawn, the foam subsided and action became normal. The dates on which foaming has occurred during the last five years are given in Table 1.

Generally, no effort was made to break up accumulations in the gas vents; on a few occasions, hose treatment was used.

Scum has not accumulated sufficiently to cause much trouble. A small quantity has been removed from a few of the gas vents, in order to reduce

the pressure on the walls. The record of removal of scum for the last three years is, as follows:

1921 ..... 48 cu. ft. from Tank No. 5
1922 ..... None
1923 ..... 36 cu. ft. from Tank No. 2
10 cu. ft. from Tank No. 3
40 cu. ft. from Tank No. 5

TABLE 1.—Data Relating to Foaming at Fitchburg, Mass.

Year.	Tank.				
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
1919	July 13 July 14	August 6 August 8	August 7 August 8	None	None
1920	None	June 14 July 2	June 14 July 15	None	None
1921	None	None	June 4	None	None
1922	None	None	None	None	None
1923	None	July 6	July 6 July 7	July 6	None

It is fair to state that difficulties due to foaming and to scum formation have been negligible throughout the nine years of operation. It appears to be possible to prevent excessive foaming by restricting the accumulation of sludge and to stop it by drawing sludge.

There has not been any odor from the tanks except that due to the exposure of the fresh sewage to the atmosphere. The sludge at Fitchburg when drawn has been practically free from objectionable odor and has contained a comparatively large proportion of solids. When applied to the sand beds, it has dried readily.

At Rochester, the Imhoff tanks of the Irondequoit Plant, first operated during March, 1917, have only once given evidence of excessive foaming. This occurred in a single tank that had been allowed to become filled to the slots with sludge. The scum in the gas vents has been removed from time to time, so that it has never risen much above the level of the sewage in the sedimentation compartments.

There has been no odor about the tanks other than that to be expected from exposure to the atmosphere of the sewage in the sedimentation compartments. The sludge has contained a large proportion of solids, has been free from objectionable odor, and has dried rapidly.

General Comparison

Although equally complete data are not available for all the plants, some information regarding the density of the sludge produced may be obtained from Table 2.

In comparing the figures in Table 2, it should be noted that the Plainfield sludge was drawn during the winter and may not have been in as good condition as during the summer season. The sludge drawn at Schenectady and

Plainfield appears to contain a much smaller proportion of solids than that from the other two plants. If it is fair to assume 7.5% solids for the former and 15% for the latter, the volume of sludge drawn at Schenectady and Plainfield would be twice as great as that at Fitchburg and Rochester for the same weight of solids.

TABLE 2.—SOLIDS IN SLUDGE AS DRAWN.

Plant.	Period.	Percentage of solids.
Schenectady, N. Y.	1918	3.3–7.3*
Plainfield, N. J.	November 21, 1922 to April 30, 1923	7.1 (average)
Fitchburg, Mass.	1922	12.85 (average)
Rochester, N. Y.		14-21

* Exclusive of one abnormally low determination.

Data are not available for an accurate comparison of the time required for the several sludges to dry to a definite extent; there have been instances at each plant of drying in as short a period as 10 days for a depth of application of about 10 in. In general, sludge beds are filled from six to ten times per year.

Consideration of all available information leads to the conclusion that the Schenectady and Plainfield plants have never functioned in a fully normal manner, as those at Fitchburg and Rochester. It should be remembered, however, that some foaming has occurred on occasions and that scum has formed to some extent at the Fitchburg and Rochester plants. These facts indicate that similar underlying conditions exist at these plants and also emphasize the possibility that there are certain differences, perhaps only in degree, which, if discovered, will explain the divergence in results obtained.

The possible causes of differences in performance of tanks at the several cities will be considered under five headings, as follows:

Character and Composition of Sewage;
Temperature of Digestion;
Preparatory Treatment of Sewage;
Kind of Biological Action;
Tank Design; and
Method of Operation.

CHARACTER AND COMPOSITION OF SEWAGE

The sewages at Schenectady and Plainfield are from separate systems, whereas those from the other cities are from combined systems.

Sewage from combined systems contains considerable quantities of solids washed by rains from the streets into the sewers. Grit-chambers have generally been provided with combined systems in order to prevent the heavy mineral detritus from entering sedimentation tanks. Such chambers vary greatly in efficiency according to their design and operation. Grit-chambers usually remove much organic and mineral detritus, but probably all allow considerable material washed from the streets to pass into the tanks. At

Fitchburg, this condition was so prevalent that it has been necessary to remove accumulations from the tanks.

The fine mineral detritus and the organic matter from the streets, when mingled with the sewage solids in the sludge compartment, may tend to weight them down and thus prevent scum formation to the same extent as in tanks receiving separate sewage. It is probable also that the volume of gas produced per pound of solids deposited is less for combined sewage. Other conditions being the same, both these facts would tend to explain the more favorable behavior of the tanks at Fitchburg and Rochester.

There does not appear to have been much difference in the comminution of suspended solids at Schenectady, Fitchburg, and Plainfield. Since the installation of fine screens at Plainfield such matter has largely been prevented from entering the tanks. The difficulties due to formation of scum which appears to be favored by a large proportion of uncomminuted solids have been troublesome at Schenectady and Plainfield, and practically negligible at Fitchburg.

The sewage reaching the plants at Schenectady and Fitchburg is comparatively fresh, possibly somewhat less so at Fitchburg, where it passes through a 30-in. inverted siphon, 5 300 ft. long, in which some solids are deposited. Decomposition of organic matter takes place to some extent in the siphon. At Plainfield and Rochester, the sewage is stale. The Schenectady and Plainfield plants have proved troublesome although they treat fresh and stale sewage, respectively. The Fitchburg and Rochester plants receive fresh and stale sewage, respectively, and yet practically no difficulty has been experienced at either plant.

Industrial Wastes.—Although at times the Fitchburg sewage has contained small quantities of paper-mill, wool scouring, and metal trades wastes, industrial wastes do not appear to have been present in sufficient quantity in this or any of the other sewages to affect greatly their composition.

Effect of Quality of Water Supply.—At Schenectady and Plainfield, the water supplies are comparatively hard. At Rochester, the water is nearly as hard as at Plainfield, and, at Fitchburg, it is distinctly soft.

The hardness of a water supply results in a large consumption of soap and in the formation of a correspondingly large proportion of insoluble soap or soap precipitate, as indicated by the results of a theoretical computation given in Table 3. These figures are based on the assumption that the same per capita daily volume of water is softened in all cases. The estimated weight of insoluble soap produced varies from 5 to 25 parts per million and is equivalent to 2.7 to 15.8% of the suspended matter in the sewage.

The soap precipitate in sewage is finely divided and of light specific gravity. It may be a factor in the production of foam and scum. In cities supplied with hard water, it obviously adds greatly to the normal quantity of suspended solids to be removed from the sewage and digested.

Strength of Sewage.—The strengths of the sewages, measured in parts per million of suspended solids, are remarkably similar at Schenectady, Plainfield, and Rochester; at Fitchburg, the sewage is considerably stronger than that at the other plants. (Table 4.)

TABLE 3.—Proportion of Insoluble Soap in Sewage.

	Schenectady.	Plainfield.	Fitchburg.	Rochester.
Sewage flow, in gallons per capita per day	92	85	89	123
Hardness of water supply, in parts per million	130	88	10	65
Water softened per pound of soap, in gallons*	36	51	210	67
Assumed volume of water softened, in gallons per capita per day	1.0	1.0	1.0	1.0
Soap consumed, in pounds per capita per day	0.0278	0.0196	0.0048	0.0149
Insoluble soap produced†, in pounds per capita per day	0.0195	0.0137	0.0034	0.0104
" " " " pounds per million gallons	211	161	38	85
" " " " parts per million	25	19	5	10
" " " " grammes per capita per day	9	6	2	5
Total suspended solids, in sewage, in parts per million	163	166	219	163
" " " " " " grammes per capita per day	57	53	74	76
Proportion of insoluble soap in suspended solids based on parts per million	15.3%	11.4%	2.3%	6.1%
Proportion of insoluble soap in suspended solids based on grammes per capita per day	15.8%	11.3%	2.7%	6.6%

* Based on Table 5, "Value of Pure Water", G. C. Whipple, p. 26.

† 0.7 lb. per lb. of soap.

The suspended solids average 55 grammes per capita in the separate sewages and 75 grammes per capita in the combined sewages. The combined sewages contain 20 grammes per capita, or 36.4% more suspended solids than the separate sewages, which may be accounted for by detritus and other matter washed from the streets during storms. This proportion is in close accord with a number of estimates made in the past.

TABLE 4.—Suspended Solids in Sewages and Deposited in Tanks.

Plant.	Year.	Population.	Flow, in million gallons per day.	Flow, in gallons per capita per day.	Suspended Solids in Sewage.		Percentage of suspended solids removed by sedimentation.	Suspended Solids Deposited in Digestion Chamber.	
					In parts per million.	In gallons per capita per day.		In parts per million.	In grammes per capita per day.
Schenectady	1922	65 000	6.0	92	163	57	71	115	40
Plainfield....	1922	40 000	3.4	85	166	53	54	89	29
Fitchburg..	1922	38 000	3.4	89	219*	74	76†	167	56
Rochester...	1922	260 000	32.0	123	163	76	44	73	34

* Due to some question as to accuracy of the figure of 266 parts per million obtained for 1922, 219 parts per million the average for the 6-year period (1915–1920, inclusive), has been substituted.

† This percentage is derived from the records for 1915 to 1920, inclusive, being substituted for the 1922 results which are open to some doubt, and includes the sludge from the humus tanks, which is pumped into the Imhoff tanks.

The quantity of suspended solids in the sewage, however, is not the only consideration, the proportion of these solids removed by sedimentation being equally important. The proportion removed varied from 44% at Rochester to 76%* at Fitchburg. Although the Fitchburg sewage contained 32% more suspended solids, in parts per million, than the Plainfield sewage, the parts per million

* Including secondary tank sludge pumped back.

of solids deposited in the tanks were 88% greater. If the volumes of sewage and the tributary populations are taken into account, it will be seen that the Fitchburg contribution is 56 grammes per capita per day, or 93% greater than that at Plainfield.

The wide difference between the number of grammes per capita per day of solids deposited in the tanks at Schenectady and Plainfield and at Fitchburg and Rochester forcibly illustrates the danger of basing the design of sludge compartments as well as other features of Imhoff tanks on a general allowance of a uniform number of cubic feet per capita. Such differences might be greatly increased by the presence of a substantial volume of certain kinds of industrial wastes in the sewage.

Temperature of Digestion

Temperature has an important influence on the rate of sludge digestion. Scarcely any determinations of temperature in digestion chambers have been made, the few available being given in Table 5.

In the absence of information on the rate of digestion in the sludge compartments of Imhoff tanks, data furnished by an experiment with a closed septic tank (Table 6) may be utilized to illustrate the effect of temperature on rate of digestion. It is recognized that this experiment was performed for another purpose and that conditions were different from those in the sludge compartment of an Imhoff tank; however, the general relation disclosed between temperature and biological action may properly serve to illustrate the principles involved in the Imhoff tank.

TABLE 5.—Temperature of Sludge in Digestion Compartments of Imhoff Tanks.

Plant.	Date.	Temperature, in degrees Fahrenheit.
Schenectady	September 6, 1923	65 to 66
	October 3, "	60
	October 10, "	62 to 63
	November 7, "	56
	November 9, "	55
Plainfield	September, "	60 to 61
Fitchburg	November 8, "	57

If the volume of gas is taken as the measure of digestion, such action was more than three times as rapid between 65° and 70° Fahr. as it was below 55° Fahr.

The relation of temperature to gas production at Worcester, Mass., is shown in Fig. 1. If this line is accepted, for the purpose of this discussion, as representing this relation, it appears that gas evolution practically ceases at 40° Fahr., that the mean annual rate of gas production is equal to that at 55° Fahr., and that at 70° Fahr. the rate of production is double the annual mean. It has been assumed that digestion proceeds at the same proportionate rate as gas production.

TABLE 6.—Gas Evolved from Septic Tank.*

Month.	Temperature of sewage, in degrees Fahrenheit.	Gas Evolved.	
		Gallons per 100 gal. of sewage.	Percentage of annual mean.
January	48.2	1.17	30
February	47.0	2.45	62
March	46.7	1.87	48
April	49.2	2.00	51
May	55.4	3.98	100
June	59.5	5.84	148
July	64.2	5.52	140
August	66.4	6.62	168
September	66.6	6.69	170
October	61.2	4.55	116
November	57.0	4.53	115
December	49.3	2.57	65
Mean	55.9	3.94	.....

* "The Action of the Septic Tank on Acid Iron Sewage", by Leonard P. Kinnicutt and Harrison P. Eddy, Fourth Report, Sewerage Commission of the State of Connecticut, 1902.

These data have been applied to the digestion of sludge in a hypothetical Imhoff tank assumed to receive from combined sewage 100 lb. of deposited solids per month, of which 60 lb. are assumed to be organic, and 40 lb. mineral matter. It has been assumed also that 30 lb., or 50%, of the organic matter must be digested, in order to produce an inoffensive sludge. This is equivalent to 30% of the total solids. For convenience, this portion of the organic matter is designated "digestible solids" although digestion is not confined to it. With separate sewage, the proportion of digestible solids would approximate 40% of the total solids.

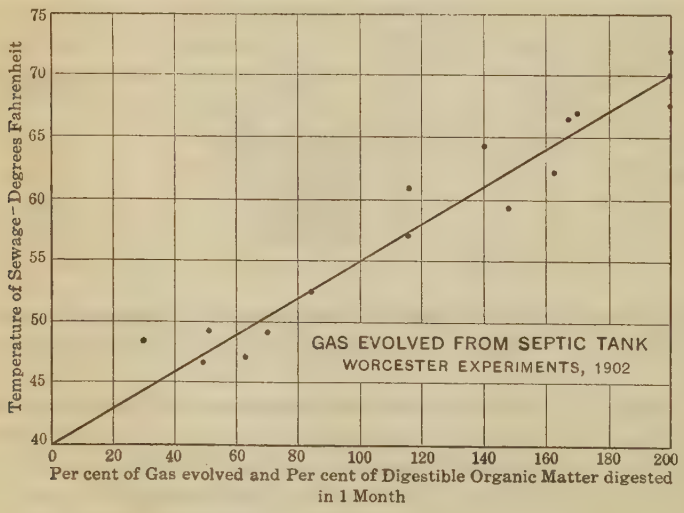


Fig. 1.

On these assumptions, the data in Table 7 show the progress of sludge digestion and accumulation at temperatures ranging from 47½° Fahr. during

TABLE 7.—THEORETICAL COMPUTATION OF SLUDGE ACCUMULATION IN IMHOFF TANKS AT TEMPERATURES RANGING FROM 47½° TO 70° FAHR.

Month.	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.
Temperature, in degrees Fahrenheit	50	47½	47½	50	55	60	65	70	67½	62½	57½	52½
Digestion of "digestible solids", percentage	68	50	50	68	100	134	168	200	184	150	118	84
Digestible solids deposited, in pounds	30	30	30	30	30	30	30	30	30	30	30	30
Residue digestible solids from previous month, in pounds	5	15	30	45	55	55	45	25	0	0	0	0
Total digestible solids in tank, in pounds	35	45	60	75	85	85	75	55	30	30	30	30
Solids digested in month, in pounds	20	15	15	20	30	40	50	60	55	45	35	25
Residue of digestible solids at end of month, in pounds	15	30	45	55	55	45	25	0	0	0	0	5
Undigestible solids contributed by sewage, in pounds	210	280	350	420	490	560	630	700	770	840	70	140
Sludge remaining in tank: Solids, in pounds	225	310	395	475	405	325	235	140	70	0	70	145
Cubic feet, average solids, 12½%	29	40	51	61	52	42	30	18	9	0	9	19
Cubic feet per capita	0.87	1.20	1.53	1.83	1.56	1.26	0.90	0.54	0.27	0	0.27	0.57

February and March to 70° Fahr. during August. These temperatures are probably as favorable as are likely to obtain in the northern part of the United States.

The digestion of digestible solids for the appropriate temperature is taken directly from Fig. 1, and expressed in percentage of the mean annual rate of digestion. The digestible solids deposited, are taken at a uniform quantity of 30 lb. per month. The residue of digestible solids from the previous months represents the quantity remaining undigested.

December is the first month, after the sludge has been completely drawn from the tank, in which there is a deficiency in digestion, such deficiency being 16% of the digestible solids deposited during that month, leaving approximately 5 lb. of residue of digestible solids on January 1. The total digestible solids subject to digestion during any month consist of the residue from the previous month plus the increment during the month.

The solids digested during the month are computed by applying the percentage digestion for the appropriate month to the digestible solids deposited in the tank each month. During months in which the digestion is 100%, or less, there is no decrease in the accumulation of digestible solids present in the tank on the first of the month. During months in which the digestion is in excess of 100%, this excess applied to the digestible solids deposited during the month gives the amount of solids digested from the accumulation of such solids in the tank at the beginning of the month. This method is adopted because the volume of gas produced in the Worcester tank is reported in terms of gallons per 100 gal. of sewage introduced into the tank during the month.

The residue of digestible solids at the end of the month is equal to the total digestible solids in the tank on the first of the month plus such solids deposited during the month minus the solids digested during the month computed as just described.

The undigestible solids contributed comprise the 40 lb. of mineral matter and 30 lb. of organic matter which need not be digested to produce an inoffensive sludge.

The sludge remaining in the tank comprises the undigested solids contributed, plus the residue of digestible solids at the end of the month, deduction being made for sludge drawn during six months, May to October, inclusive, at a uniform rate per month, sufficient to remove all sludge at the end of October, except a small quantity constituting a circulating load which does not enter into the computations. The solids drawn amount to 140 lb. per month, or 840 lb. per year, 360 lb. having disappeared as the result of digestion.

The cubic feet of sludge remaining in the tank has been computed on the assumption that the sludge will contain 15% of solids at the bottom, 10% at the surface, and 12½% as an average.

The hypothetical data have been reduced to terms in common use by assuming that the solids deposited in the tank correspond to 3 lb. per capita per month. For domestic sewage, this would approximate 2.2 lb. per capita, per month. This computation indicates a required capacity for sludge storage

of 1.83 cu. ft. per capita, the maximum quantity being stored at the end of April. To this must be added an allowance for the "neutral zone" between the surface of the sludge and the horizontal plane through the slots, say, 20%, making a total required capacity of 2.20 cu. ft. per capita.

The total sludge drawn per year, assuming 15% solids, is equivalent to 2.715 cu. ft. per capita.

The rate of digestion and the per capita sludge accumulation, based on temperatures 5° lower than those used in computing Table 7, are given in Table 8.

TABLE 8.—Theoretical Computation of Sludge Accumulation in Imhoff Tanks at Temperatures Ranging from 42½° to 65° Fahr.

Month.	Temperature, in degrees Fahrenheit.	Sludge remaining in tank, in cubic feet per capita.
January	45	0.96
February	42½	1.32
March	42½	1.71
April	45	2.04
May	50	1.77
June	55	1.44
July	60	1.11
August	65	0.72
September	62½	0.33
October	57½	0.00
November	52½	0.30
December	47½	0.63

This computation indicates that a decrease of 5° in temperature results in an increase of maximum sludge accumulation from 1.83 to 2.04 cu. ft. per capita, which, with a 20% allowance for the neutral zone, requires an increase in sludge storage capacity from 2.20 to 2.45 cu. ft. per capita, not allowing for storage of any solids in the scum compartment. This is an increase of slightly more than 10%, which would be greater if the difference between the assumed schedules of temperature were greater. It may be that in practice the differences are considerably greater than those assumed.

Curves *A* and *B*, Fig. 2, illustrate the variation in rate of digestion and evolution of gas, with temperature. The full line, Curve *B*, shows the rate of digestion, based on the assumption that sludge is drawn, thus reducing the quantity of digestible matter in the tank and, therefore, also the rate of digestion; the dotted line shows the rate of digestion on the assumption that sludge is not drawn, therefore, the rate of digestion is greater than that shown by Curve *B*.

At the lower temperature (Curve *A*), there is always an excess of digestible matter. Therefore, the drawing of sludge does not affect the rate of digestion. Curves *C* and *D* show the rate of accumulation of sludge by volume at the different temperature schedules assumed, and the effect of regular uniform removal of sludge from May to October. The importance of beginning to draw sludge as early as inoffensive material can be obtained, and at a proper rate, is readily seen. The dotted lines, Curves *C* and *D*, show the rate of accumulation if sludge is not drawn.

It should be noted that, on the basis of the assumed schedule of operation, the maximum accumulation of sludge occurs at the end of April, and that the maximum volume of gas is generated at the end of August.

For the temperature schedule, 47.5° to 70° Fahr., the sludge chamber should be proportioned to accommodate 1.83 cu. ft. per capita of sludge below the neutral zone. Curve *D* shows that the sludge accumulation at 42.5° to 65° Fahr., will occupy nearly one-half of such a neutral zone.

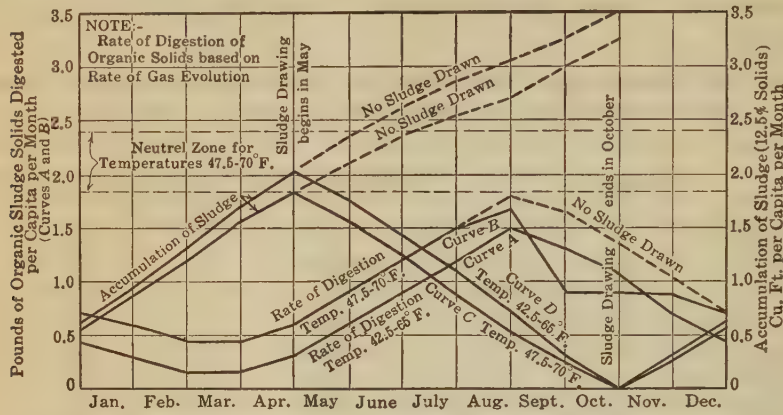

NOTE:-
Rate of Digestion of Organic Solids based on Rate of Gas Evolution
Neutral Zone for Temperatures 47.5-70° F.
Sludge Drawing begins in May
Sludge Drawing ends in October
No Sludge Drawn
No Sludge Drawn
No Sludge Drawn
Accumulation of Sludge
Rate of Digestion Temp. 47.5-70° F.
Rate of Digestion Temp. 42.5-65° F.
Curve B
Curve A
Curve C Temp. 47.5-70° F.
Curve D Temp. 42.5-65° F.
Pounds of Organic Sludge Solids Digested per Capita per Month (Curves A and B)
Accumulation of Sludge (12.5% Solids) Cu. Ft. per Capita per Month (Curves C and D)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.

FIG. 2.—ACCUMULATION OF SLUDGE AND RATE OF DIGESTION OF ORGANIC SOLIDS IN IMHOFF TANKS.

It will be noted that, if the sludge is not drawn according to the assumed schedule, the neutral zone will soon become entirely filled and, also, that the excessive sludge accumulation will occur when gas is being produced at a high rate. This combination accounts for much of the foaming that has occurred at many plants. It is easy to understand also why the drawing of sludge frequently causes foaming to cease.

The temperature during the summer months is of more importance than the yearly average temperature, because during these months the largest quantity of solids must be digested. It is probable that an unusually low temperature in the cool part of the year, together with an exceptionally high temperature in a short warm season, may result in an abnormally large accumulation of digestible solids during the cool period, followed by very violent digestion during the warm period. This combination of conditions might bring about the excessive foaming that has occurred at various plants during the warm months. Winter foaming, such as was experienced at one time at Plainfield, is relatively rare, but demonstrates the possibility of obtaining conditions that will offset the inhibitory effect of low temperatures.

Data are not available for a comparison of temperatures in the sludge compartments of the several tanks under consideration. Such determinations are needed before definite conclusions can be made relative to the effect of temperature in explanation of the differences in tank behavior. There can be no doubt, however, that temperature should be considered in the determination of the required capacity of digestion compartments.

Preparatory Treatment of Sewage

At Schenectady, about 75% of the sewage passes through coarse racks and centrifugal pumps, on the way to the treatment plant, at which all the sewage passes through single racks consisting of 2 by $\frac{5}{16}$-in. bars spaced 1½ in. in the clear. No grit-chambers are provided.

At Plainfield, the sewage received only coarse screening at the time of maximum trouble with foaming. Later, a Riensch-Wurl screen having $\frac{1}{16}$-in. slots was installed, since which less difficulty with foaming has been experienced.

At Fitchburg, the sewage is screened through coarse racks, with a clear space of 1¾ in., and passes through grit-chambers. A small proportion is pumped.

At Rochester, the sewage passes through coarse racks, grit-chambers and Riensch-Wurl screens having slots ⅛ in. in width. During storms, a part passes through a screen having ¼-in. slots.

At Rochester, fine screening may account for the freedom from trouble due to foaming and forming of scum; it certainly has been of assistance in these respects at Plainfield. At the latter plant, however, the formation of scum and foam is still excessive.

Obviously, the quantity of organic matter to be digested can be somewhat reduced by fine screening. Gas is more easily liberated from gas-lifted masses of finely divided solids than from masses of coarse solids, which tend to interweave and hold together. Fine screening, therefore, may aid greatly in reducing scum formation. In the light of experience at Fitchburg, however, fine screens cannot be said to be a necessity under all conditions, and the advisability of installing them would appear to depend on the expense involved in disposing of the coarser parts of the suspended solids by fine screening on the one hand, and by tank treatment on the other. There is no doubt that preparatory treatment by fine screening affords a factor of safety in the operation of the tanks.

Kind of Biological Action

As stated previously, biological action increases and decreases greatly with the rise and fall of temperature. The effect of other factors, however, are important, such as adequacy and character of food supply, moisture, oxygen, inorganic poisons, light, antagonistic organisms, waste products of biological activity, and reaction of environment.

Ample food supply and moisture are always present in the sludge compartment of an Imhoff tank. Although evidence is lacking, there is a possibility that different sewages produce sludges of such different composition that corresponding variations are produced in the predominant organisms, with resultant variation in the kind of biological action.

Light, oxygen, and inorganic poisons are of relatively small importance in the processes of digestion. The circulation of liquid between the digestion and the sedimentation chambers and the periodic withdrawal of sludge largely bring about the removal of the products of decomposition, and it is

probable that the "self-poisoning" of the essential organisms in the sludge is not an important factor.

The reaction of the sludge and liquid in the digestion chamber may have a marked effect on the digestion processes. It is difficult to state, however, whether excessive acidity with poor quality of sludge is a cause or a symptom. In certain instances, notably at the Pennypack Creek plant at Philadelphia, Pa., the use of lime is said to have resulted in marked improvement in the behavior of Imhoff tanks. It is also possible that large quantities of carbon dioxide may have a distinct inhibitory effect on biological activity. This seems more significant in the case of scum in which the gases are entrapped, than in sludge from which they can more readily escape.

With scum, however, the substitution of gas for moisture in the voids may render it less easily broken by organic activity than is the case with wet sludge. Furthermore, there is much less opportunity for washing out the final products of biological activity and the correspondingly greater opportunity for self-poisoning of the organisms of decomposition. These conditions are particularly favored by allowing the scum to remain undisturbed for long periods, as was done at Schenectady, during 1922.

No data are available by which the kinds of biological action at the several plants under consideration can be compared, with a view to determining the causes of the differences in behavior. It seems reasonable to expect a difference between the kind of action in the sludge and that in the scum. It is conceivable that the tendency to form scum may be largely due to the physical characteristics of the tanks and to the degree of comminution of the coarser solids. Any difference in the predominant kind of biological action in the scum, may account for the phenomenon of foaming.

In general, two kinds of decomposition develop: First, the production of carbon dioxide by the splitting of non-nitrogenous organic matters, or fermentation; and, second, the production of methane by the splitting of nitrogenous organic matters, or putrefaction. Both kinds of decomposition may be included in the digestion process.

The initial decomposition of solids in a digestion compartment results in the production of a large proportion of carbon dioxide, due to the fact that fermentation appears to start at a rapid rate. Under favorable conditions, however, the character of decomposition quickly changes and methane becomes the predominating gas produced, roughly, 85% of the total volume. These stages of decomposition appear to be like those observed in the septic tank, as demonstrated by the experiments at Worcester and elsewhere. Unsatisfactory results will be obtained if biological conditions, such as food and environment do not permit the change from carbon-dioxide fermentation to methane decomposition.

It has frequently been suggested that scum and foam formation, which recently at certain plants has become more troublesome than ever before, may be due to acid fermentation of large quantities of mash from the illicit manufacture of alcoholic beverages. There is ample evidence of the presence of large quantities of such materials in sewage and it is probable that they

are peculiarly susceptible to fermentation. Whether or not this is an important factor in the formation of gas and scum, such material has been very noticeable in the excessive scum found at some plants.

It may be that certain kinds of action, such as fermentation with production of large quantities of carbon dioxide, do not as rapidly disintegrate, gasify, and render soluble the coarser organic substances as other varieties of action. Therefore, with fermentation, for example, there might be an abnormal accumulation of scum-forming solids tending to overload the sludge compartment or to form gas-lifted masses in such quantities that excessive scum formation is inevitable.

There is little information about the biological action in digestion compartments. Practically nothing is known of enzymes, their origin and functions, and the conditions favoring their action. The effect of higher forms of organisms, possibly antagonistic to the digestive varieties, is also little understood. The agitation of scum for the liberation of gas gives promise of confining the action within the digestion compartment to that which is typical of sludge and thus avoiding difficulty in undesirable biological action which may take place in the more or less compact scum. Although it may be possible to obtain satisfactory digestion in shallow tanks in this manner, the indications are that a much larger sludge compartment will be required in such tanks to offset the greater volume of sludge and the tendency for scum formation.

Sanitary engineers will look forward with keen interest for the results of the investigations being conducted jointly by the New Jersey Agricultural Experiment Station, New Jersey State Board of Health, and a committee of two sanitary engineers, a chemist, and a practical operator, all members of the New Jersey Sewage Operators' Association, serving as an Advisory Committee.

Tank Design

The physical characteristics of the tanks are illustrated by Fig. 3. Comparisons are made on the basis of populations served and flows treated during 1922 and not on populations and flows for which tanks were designed.

The tanks differ in length, from 64 ft. at Plainfield, to 110 ft. at Rochester. The fact that most of the solids settle comparatively near the inlet end and in materially decreasing quantities farther inside should be considered in determining the length of tank. On this account, a comparatively short tank may be more advantageous than a long one.

The digestion space in the tanks at Schenectady is subdivided by partition walls into eight compartments, and that at Plainfield into five compartments. There are no subdivisions of the digestion compartment in the Fitchburg plant, although buttresses are used in the upper part of the side scum compartments. The digestion space of the Rochester plant is subdivided into three compartments by partition walls.

It is important to distribute the sludge throughout the sludge storage space as uniformly as possible, in order that each part of the tank may fully serve its allotted purpose and that no part may be overloaded. During

the early days of operation at Plainfield it was found that more than half the sludge was collected in the first hopper, and a large part of the remainder in the second hopper; reversal of flow resulted in a similar distribution of sludge in the hoppers at the other end of the tank. Little or no sludge was collected in the middle hopper.

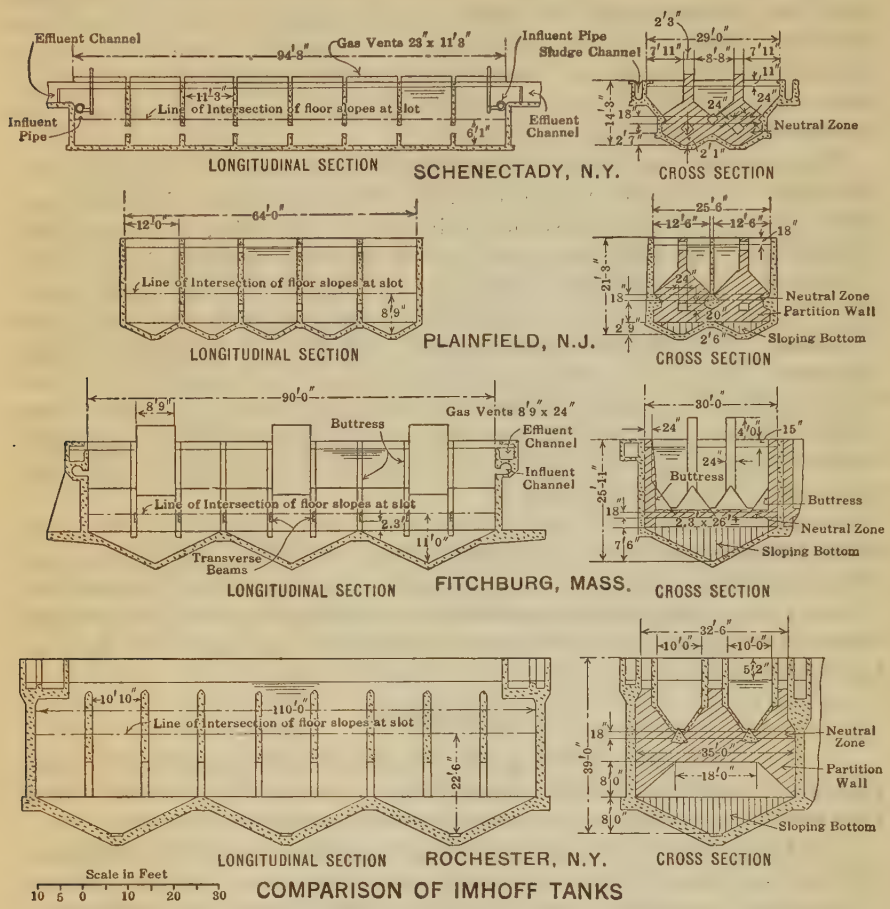


FIG. 3.

Reversal of flow in multiple-hopper tanks assists in the distribution of solids. Further aid in equalizing the accumulation of sludge may be obtained by permitting it to flow from one hopper to another by gravity. At Schenectady and at Plainfield, the only opportunity for the sludge to flow from one hopper to another is through the small opening in each of the partition walls. As these openings are only about 2 ft. square (they are smaller at Plainfield) and are some distance above the bottom of the tank, the opportunity for equalization is not good, particularly in the plant at Schenectady in which the sludge must pass through three such openings to reach one of the central compartments.

At Fitchburg, the opportunity for the passage of sludge from one compartment to another is much better, the only interfering members being the transverse beams and side-wall buttresses. The elevation of the beams practically coincides with that of the assumed neutral zone. The most restricted space for the flow of sludge is between the ridge separating the hoppers and the transverse beam, and is approximately 2.3 by 26.0 ft. in vertical cross-section.

At Rochester, a large opening between the sludge compartments affords ample communication.

Distribution of Solids Among Tanks.—It is as important to obtain uniform distribution of solids among the several tanks as within the hoppers of a single tank, but this is difficult to obtain, as has been demonstrated at Schenectady, where it is evident from observation and analyses that one battery of tanks receives sewage containing a much larger quantity of suspended matter and coarser solids than the other. A similar, although less marked, segregation of solids, has occurred at Fitchburg.

Depth of Tanks.—The depths of the tanks below the surface of the sewage, and the distances below and above the bottom of the assumed neutral zone, are given in Table 9.

An essential difference between the good and bad-acting tanks is in their respective depths. The Schenectady and Plainfield tanks are much shallower than those at Fitchburg and Rochester. The influence of depth of tank on the action within it is not clear. One theory is, that in the deeper tanks there is a longer path through which the gas-lifted solids must ascend in order to form an accumulation of scum at the top, and that this prolonged travel affords opportunity for the escape of the gases after which the solids may return to the bottom, thus avoiding their accumulation as scum.

TABLE 9.—Depths of Tanks and Compartments.

Plant.	Maximum depth, in feet, below surface of sewage.	Maximum distance, in feet, below bottom of neutral zone.	Maximum distance, in feet, above bottom of neutral zone.
Schenectady	13.8	4.6	9.2
Plainfield	19.8	7.25	12.55
Fitchburg	24.7	9.5	15.2
Rochester	33.9	21.0	12.9

When the sludge has accumulated to the level of the bottom of the neutral zone, the maximum possible length of travel is greater at Fitchburg and Rochester than at Schenectady. At Plainfield and Rochester, the lengths of travel are practically the same. It is perhaps more significant that during most of the year, when the accumulation of sludge should be considerably below the bottom of the neutral zone, particularly at Fitchburg and Rochester, the length of travel at these plants is much greater than that at Schenectady and Plainfield.

In the deeper tanks, the gas is generated under greater pressure and the bubbles entrained in the sludge are subject to considerable expansion on

rising, thus possibly aiding in their escape from the solids; likewise, there is a deeper accumulation of sludge, which tends toward greater density.

The maximum possible depths of sludge at Fitchburg and at Rochester are approximately twice and four times, respectively, as great as that at Schenectady.

The greater depths of the tanks at Fitchburg and Rochester appear to explain in part the absence of excessive scum formation and the comparatively greater density of sludge at these two plants.

Sludge Compartments.—Dr. Imhoff's allowances for capacity of sludge compartments for separate and combined sewages are 1.2 and 1.8 cu. ft. per capita.* This allowance of 50% extra capacity for combined sewage is somewhat in excess of the extra quantity of solids in the sewage of Fitchburg and Rochester, which was 36%, but is probably necessary because of the greater proportion of undigestible solids in the combined sewage. It appears that, for the present population, the tanks have capacities in excess of those recommended by Dr. Imhoff (Table 10). The compartments at Rochester are 43% in excess of his allowance.

Dr. Imhoff has pointed out the necessity of having a larger sludge compartment for a shallow than for a deep tank. It has been shown that the sludge drawn at Schenectady and Plainfield contains only about 7½% solids, or one-half the proportion in that drawn at Fitchburg and Rochester. The volume of sludge at Schenectady and Plainfield, therefore, would be twice that at Fitchburg and Rochester for an equal weight of solids. If it is reasonable to assume that the Schenectady and Plainfield sewages would produce a sludge containing 15% solids in deep tanks, such as those at Fitchburg and Rochester, the allowance for volume of sludge to be stored in the Schenectady and Plainfield tanks as built should be doubled on account of their shallowness. The sludge capacities provided at Schenectady and Plainfield are only slightly more than one-half that required by this reasoning.

TABLE 10.—Capacity of Sludge Compartments.

Plant.	Dr. Imhoff's recommended allowance, in cubic feet per capita.	Dr. Imhoff's allowance modified because of shallowness of tanks, in cubic feet per capita.	Actual gross capacities, in cubic feet per capita.	Actual net capacities after deducting neutral zone, in cubic feet per capita.
(1)	(2)	(3)	(4)	(5)
Schenectady	1.2	2.4	1.40	1.02
Plainfield	1.2	2.4	1.49	1.22
Fitchburg	1.8	1.8	1.88	1.47
Rochester	1.8	1.8	2.49	2.21

Although it has been found to be convenient to consider the capacity of sludge compartments in terms of gross capacity, it is important to recognize that it is not wise to depend on utilizing the entire compartment for the storage of sludge.

* *Engineering News*, Vol. 75 (1916).

Doubtless, the depth of the neutral zone should be determined from a consideration of several conditions, such as the depth of the sludge accumulation and proportion of uncomminuted solids. Probably it should be deeper in shallow tanks treating fresh unscreened sewage than in deep tanks treating stale screened sewage. In this study, however, it seems reasonable to allow the same depths in all cases, namely, 18 in. In order to arrive at the proper net available storage capacity, the effect of deducting the volume of the neutral zone from that of the gross compartments, depends on the design of the tank. Thus, making this deduction, the gross capacities are reduced approximately 11% at Rochester and 27% at Schenectady.

The load on the digestion compartment may be expressed in pounds of suspended solids deposited per year per cubic foot of its capacity. A comparison of the loads computed in this manner, based either on the gross or the net capacity of the sludge compartment (Table 11), shows that the load at Rochester is the smallest and that at Fitchburg the greatest. At Schenectady and Plainfield, the loads are approximately 1.5 and 2.5 times as great as that at Rochester, if deduction is made for the neutral zone.

These loads are based on the quantity of solids deposited without any reference to their partial digestion or to the volume of sludge formed by them. As the solids lie in the tank, they are partly digested and allowance should be made therefor. During the spring, at times of maximum load, it is not likely that more than one-half the digestion has taken place. Assuming that digestion has progressed that far, 20% of the solids in the separate sewage and 15% in the combined sewage, have disappeared leaving 80% and 85%, respectively. If allowance is made for digestion and if the large volume of thin sludge at Schenectady and Plainfield proportionately limits the load which can be placed on the sludge compartments, the relative loads by volume on the sludge compartments at the various plants, calling that at Rochester unity, are shown in Table 12.

TABLE 11.—Loads upon Sludge Compartments.

Plant.	Suspended solids based on gross sludge capacity, in pounds per year per cubic foot.	Ratio to load at Rochester.	Suspended solids based on net sludge capacity, in pounds per year per cubic foot.	Ratio to load at Rochester.
(1)	(2)	(3)	(4)	(5)
Schenectady....	21.9	1.94	30.0	2.44
Plainfield.....	15.5	1.37	18.9	1.54
Fitchburg.....	24.1*	2.13	30.9*	2.51
Rochester.....	11.3	1.00	12.3	1.00

* Includes secondary tank sludge pumped to Imhoff tanks.

It appears that the load at Schenectady is 4.6 times as great per cubic foot as that at Rochester. If the load on the tanks at Rochester is assumed to be correct, the Schenectady tanks are greatly overloaded.

Of course, the Rochester sludge compartment may be too large, in which case, the relative loads by volume would make the compartments at Schenectady and Plainfield appear to be more inadequate than they actually are.

If the relative load by volume at Fitchburg is assumed as 1, the loads by volume at Schenectady and Plainfield become 1.8 and 1.2, respectively, and that at Rochester becomes 0.4. It is not the purpose in this paper, however, to consider the proper load by volume, but rather to compare existing conditions at the several plants in order to arrive at an explanation of the differences in behavior of tanks.

TABLE 12.—Relative Load by Volume Based on Actual Net Volume of Sludge Compartments.

Plant.	Relative load by volume.
Schenectady	$\frac{30.0 \times 0.80 \times 2}{12.3 \times 0.85} = 4.6$
Plainfield	$\frac{18.9 \times 0.80 \times 2}{12.3 \times 0.85} = 2.9$
Fitchburg	$\frac{30.9 \times 0.85 \times 1}{12.3 \times 0.85} = 2.5$
Rochester	$\frac{12.3 \times 0.85 \times 1}{12.3 \times 0.85} = 1.0$

Although these relative loads by volume may be based on a rather uncertain combination of assumptions, they are, nevertheless, significant. The larger loads at Schenectady and Plainfield, as compared with those at Rochester and Fitchburg, may explain in part the difficulties of operation at the former places. If the load on the sludge compartment is too great, the sludge must either be drawn before it has become thoroughly digested and is, therefore, offensive, or be allowed to accumulate, thus encroaching on the neutral zone, or both. The latter condition is particularly favorable to scum formation.

Scum Compartments.—The largest per capita volume of scum compartments is at Fitchburg, those at Schenectady and Plainfield being only about one-half as large.

TABLE 13.—Scum Compartments.

Plant.	Volume, in cubic feet per capita.	Loading, in pounds per year per cubic foot.
Schenectady	0.76	40.4
Plainfield	0.72	32.1
Fitchburg	1.59	28.5
Rochester	0.55	49.7

If the tanks were functioning under ideal conditions, that is, with gas entirely liberated as the solids rise, and all the solids in the form of sludge, there would be little logic in considering the load on the scum compartments. Under existing conditions, however, with such large accumulations of scum, the quantity of solids tributary to the scum compartments should be considered. For the sake of comparison, the loading has been computed on the basis of the entire weight of solids deposited in the tanks, no effort being made to sub-

divide the solids into those parts which may constitute sludge and scum, respectively (Table 13). The smallest load is found to be at Fitchburg, and the greatest at Rochester. At Rochester, however, the coarse solids have been removed by screening, which may account for the small accumulation of scum, notwithstanding the apparent heavy load. The excess of load at Schenectady, over that at Fitchburg, indicates one reason for the trouble due to scum at the former plant.

The load on the scum compartments at Plainfield is somewhat greater than that at Fitchburg. Taking into consideration the shallowness of the Plainfield tanks—a condition favorable to scum formation—the load may be great enough to account in part for the trouble due to scum, notwithstanding the screening of the Plainfield sewage.

Some engineers have maintained that small gas-vent area is advantageous, whereas others have argued that the area should be large. Although the areas of the gas vents are substantially equal in all the plants, those at Rochester comprise a much larger proportion of the total tank area than the others. This does not tell the whole story, however. A more logical method of studying the effect of gas-vent areas would seem to be to compare them with the volume of tributary sludge space, or better yet, with the loading in pounds of sludge solids, as given in Table 14.

TABLE 14.—Gas-Vent Areas.

Plant.	Area of gas vents, percentage of total tank area.	Area of gas vents, in square feet per cubic foot of sludge space.	Loading, in pounds of solids per year per square foot of gas-vent area.
Schenectady	14.8	0.034	644
Plainfield	14.3	0.023	674
Fitchburg	15.0	0.031	777 { side vents 335 central vents 2040 }
Rochester	26.8	0.0133	849

On these bases of comparison, the gas vents at Rochester are much less liberal than the others.

The load of solids and probably, therefore, the volume of gas tributary to the vents, is much greater at Fitchburg and Rochester than at Schenectady and Plainfield. The absence of trouble at the former plants tends to support the claim that small vents are preferable, although the differences in loads cited are not very great. At Fitchburg, however, there is a marked difference between the equivalent area of the side vents and of the central vents, the latter being nearly six times as heavily loaded as the former. Experience at this plant does not indicate a great difference in behavior between the side and central vents. There has been slightly more foaming in the latter. The evidence furnished by the plants under consideration is not sufficient to justify a conclusion in regard to the comparative merits of the large and the small gas-vent areas.

The theory has been advanced that gas-lifted sludge hitting against the inclined bottom of the sedimentation compartment may thus lose, through

agitation, a part of its gas. Thus, a design like that at Schenectady, which makes most of the sludge directly tributary to the scum compartment, may be less advantageous than the Fitchburg design in which a larger proportion of the gas-lifted sludge must roll or slide a considerable distance on the overlying concrete.

Another theory is that gas-lifted sludge held in the inverted V-trough beneath the sedimentation compartment, as in the central scum space at Fitchburg, keeps the solids wet and permits more active digestion than in the gas vents where a greater proportion of the gas may escape more quickly and a compact mass of scum be formed, as at Schenectady.

Operation

Imhoff tanks should be operated in such manner as best to develop their two important functions: The removal from sewage of suspended solids and their digestion to a sufficient degree to render them inoffensive when spread on drying beds and used for filling or fertilizer, or finally disposed of otherwise.

Efficiency of sedimentation should be controlled as far as practicable with the aid of determinations of the proportion of suspended solids removed from the sewage during its passage through the sedimentation compartment. It should be recognized that digestion of organic matter results from biological action, and that operating conditions should be made as favorable as possible to such action.

All the solids entering the digestion compartment must be digested or removed in the form of sludge or scum. It is of the utmost importance for the operator to know from time to time to what extent solids have accumulated in the digestion compartment in order that he may compare the quantity of such accumulation with the capacity provided for it. In some instances, this can be determined by the measurement and analysis of sludge and scum, but, in others, this has proved impracticable, if not impossible. Where this method is impracticable, helpful information can be obtained from analyses of tank influent and effluent and measurement of volume of sewage treated, together with data relating to sludge drawn.

The operator should carefully study the condition of sludge available for drawing, particularly during the early spring, in order to determine the degree of digestion that has taken place, and govern the drawing of sludge accordingly, giving due consideration to the relation between the volume of sludge on hand and the capacity of the digestion chamber. The drawing of sludge in the spring should be postponed as long as practicable in order to reduce the danger of removing offensive smelling sludge. On the other hand, it must not be allowed to accumulate to the extent of causing foaming or other difficulty.

Intelligent and skillful plant performance is greatly aided by certain analytical records which enable the operator to know what is being accomplished by the tanks and how their efficiency can be increased. The volume of sewage should be accurately measured. Estimating the tributary population from year to year will enable the operator to compile some of his analytical

and load data on a per capita basis, which is very helpful in some instances. The sludge and scum removed from the tanks should be accurately measured and the solids determined. It is also desirable to determine the proportion of organic matter in the sludge and scum removed. From measurements and analyses, the proportion of the solids which disappear through digestion should be computed. Determinations of the temperature in the sludge compartment will aid in understanding the conditions within the compartment.

Satisfactory results in the operation of such tanks are as dependent on intelligence and skill in operation as on good design. All the plants under consideration have had technical supervision and the differences in results obtained cannot be attributed to lack of skillful operation.

Conclusions

This study of the available evidence afforded by the Imhoff tanks at these four plants, summarized in Table 15, leads to the following conclusions:

1.—The mineral and heavy relatively stable organic matter of the combined sewage at Fitchburg and Rochester may tend to prevent the formation of excessive scum and foam by weighting down the sludge.

2.—Coarse and uncomminuted solids at Schenectady are probably an important factor in the excessive formation of scum.

3.—If the water supply is hard, the insoluble soaps formed constitute a substantial increment in the suspended solids of the sewage and may favor the formation of foam and scum.

4.—The variation in the quantity of suspended solids to be removed from different sewages is so great that the design of the digestion compartment should be based on the quantity of solids to be deposited in it, rather than on a general assumption of a definite number of cubic feet per capita.

5.—Temperature is a factor of fundamental importance in the digestion process.

6.—The required capacity of the digestion compartment is governed largely by the available temperature and by the duration of the period of low temperature.

7.—Difficulties will be minimized by drawing sludge as early in the spring as inoffensive material can be obtained, by continuing the drawing at a rate sufficient to provide as small an accumulation in the sludge compartment as practicable during hot weather, and by removing before cold weather all sludge except that required for seeding.

8.—Fine screening by reducing the quantity of digestible solids to be deposited from the sewage, reduces the load on the digestion compartment correspondingly, and by removing the coarser matter particularly favorable for scum formation tends to reduce difficulty from that source.

9.—Fine screening appears to have been beneficial at Plainfield and Rochester; although not a necessity, it affords a factor of safety in the operation of Imhoff tanks.

10.—The advisability of installing fine screens appears to depend on the relative cost of disposing of the coarser part of the suspended solids by fine screening on the one hand and by tank treatment on the other.

11.—It is important to distribute the deposited solids as uniformly as possible throughout the digestion compartment; for this reason a relatively short tank is advantageous.

TABLE 15.—Summary of Essential Data on Imhoff Tanks.

Item.	Schenectady, N. Y.	Plainfield, N. J.	Fitchburg, Mass.	Rochester, N. Y.
Tributary population (1922)	65 000	40 000	38 000	260 000
Character of Sewage:				
Flow, in million gallons per day	6	3.4	3.4	32
Separate or combined sewers	Separate	Separate	Combined	Combined
Strength (suspended solids), in parts per million	163	166	219	163
Freshness	Fresh uncomminuted	Stale	Fresh	Stale
Industrial wastes	Practically none	Practically none	Small amount	Small amount
Hardness of water supply, in parts per million	130	88	10	65
Temperature, in degrees, Fahrenheit	Av. (1922) 56.1 Max. 64.7	46–70	No data	Av. 54
Preliminary Treatment:				
Screening	Coarse rack	Fine screens, 1/16-in. slot	Coarse racks	Fine screens, 1/8-in. slots
Grit-chambers	None	None	Yes	Yes
Imhoff Tanks:				
Sedimentation period, in hours	3 3	3.4	6.4	1.1
Depth of Tanks:				
Total maximum water depth	13 ft. 9 in.	19 ft. 9 in.	24 ft. 8 in.	33 ft. 10 in.
Sludge Compartment:				
Depth below plane of slots	6 ft. 1 in.	8 ft 9 in.	11 ft. 0 in.	22 ft. 6 in.
Depth below 18-in. neutral zone	4 ft. 7 in.	7 ft. 3 in.	9 ft. 6 in.	21 ft. 0 in.
Distance of lowest point of overflow below 18-in. neutral zone	2 ft. 6 in.	2 ft. 0 in.	2 ft. 0 in.	13 ft. 0 in.
Depth below overflow to adjacent compartment	2 ft. 1 in.	5 ft. 3 in.	7 ft. 6 in.	8 ft. 0 in.
Cubic feet per capita	1.40	1.49	1.88	2.40
Number of hoppers	8	5	3	3
Ease of intercommunication	Poor (2-ft. square opening)	Poor (opening 20 by 24 in.)	Good	Good
Scum Compartment:				
Cubic feet per capita	0.76	0.72	1.59	0.55
Area, gas vents; percentage of tank area	14.8	14.3	15	26.8
Area, gas vents; square feet per cubic foot sludge capacity	0.034	0.023	0.031	0.0133
Loading, Deposited Solids:				
Pounds per year per cubic foot sludge and scum space	14.2	10.4	13.1	9.2
Pounds per year per cubic foot sludge space	21.9	15.5	24.1	11.3
Pounds per year per cubic foot sludge space after deducting 18-in. neutral zone	30.0	18.9	30.9	12.3
Pounds per year per cubic foot scum space	40.4	32.1	28.5	49.7
Pounds per year per square foot gas-vent area	644	674	777	849

12.—Digestion compartments should be subdivided as little as practicable, and liberal opportunity should be afforded the sludge to spread uniformly from one end of the tank to the other.

13.—Lack of uniform distribution of sludge throughout the digestion compartment may have been an important factor in the unfavorable action at Schenectady and Plainfield.

14.—Frequent reversal of flow is necessary for the successful operation of multiple-compartment tanks.

15.—It is important to obtain nearly equal distribution of solids among the several tanks; failure to accomplish this at Schenectady has been a factor in the difficulty of operation.

16.—There appears to be a decided advantage in the greater depth of tanks at Fitchburg and Rochester in preventing excessive scum formation and in providing sludge with a comparatively large proportion of solids.

17.—In the design of the digestion compartment consideration should be given to the probable density and corresponding volume of the sludge as it will lie in the tank.

18.—If tanks must be shallow, substantial additional capacity must be provided in the digestion compartments.

19.—Taking into account both the load in pounds of deposited solids and the probable density of the sludge, the digestion compartments at Schenectady and Plainfield appear to be relatively much smaller than those at Fitchburg and Rochester, a condition offering one explanation of the difficulties at the former plants.

20.—Evidence is so meager and conflicting that a final conclusion regarding the comparative merits of a small and a large proportion of gas-vent area is not justified.

21.—Any opportunity for the breaking of rising gas-lifted sludge, as by impact on the floor partition between the sedimentation and digestion compartments, may be of advantage.

22.—Successful operating results depend as much on intelligence and skill in operation, as on correct design.

23.—Complete accurate operating data are highly desirable, and very helpful to intelligent and skillful operation.

24.—Little is known about the kind of organisms, or the biological action in Imhoff tanks. The absence of such information has made it necessary to confine this study to the structural, physical, and chemical differences. They appear to explain many results without consideration of biological action. The latter may have been a factor, possibly as a result of certain structural features, rather than as a primary cause of unfavorable action.

25.—The differences in results obtained are not explained by a single condition, but appear to be due to many factors which in the aggregate make a wide difference between the two pairs of plants. The essentially unfavorable conditions at Schenectady and Plainfield are:

(*a*) Shallowness of tanks;
(*b*) Inadequate digestion compartments;
(*c*) Impossibility of uniform distribution of sludge throughout digestion compartments;
(*d*) Large number of digestion compartments, making it impracticable to determine volume and density of sludge in them and difficult to cope with sludge and scum problems;
(*e*) Absence of heavy solids from street washings;
(*f*) Relatively large proportions of insoluble soaps.

Need for Research

Additional information is much needed on both the biological and physical phases of the process of digestion, as to the:

1.—Temperatures which prevail in digestion compartments throughout the year in different parts of the country, preferably determined by recording thermometers which should be placed both in scum and sludge compartments.

2.—The reaction, or hydrogen-ion concentration of sewage, liquid in digestion compartment, sludge, and scum.

3.—The rate of digestion at different temperatures, when all other conditions are controlled so that they are as nearly uniform as possible, in the parallel tests.

4.—Variation in the volume of gases evolved at different temperatures, with the object of utilizing such determinations as an index of the rate of digestion.

5.—Variation in the composition of gases evolved at different plants and temperatures, with a view to utilizing the composition of the gas as an index of the character of decomposition in progress.

6.—Kind and functions of the organisms predominating in sludge and scum compartments, respectively, under varying conditions, such as different reactions and temperatures.

7.—The effect of different depths, by operating tanks in exact parallel in all other respects including the loads.

8.—The effect on permissible load, of removing the coarse solids from the sewage before it is introduced into the tanks, determined by tests in exact parallel in all other respects.

9.—The effect on permissible load, of continuous artificial agitation of scum to an extent which will cause the solids to remain in the sludge mass practically all the time.

Many other tests might be suggested, but those enumerated appear to be particularly promising. In order that results may be as conclusive and valuable as possible, it is of the utmost importance that, in parallel tests, the variables be reduced, preferably to a single condition, such as temperature, depth, or removal of the coarser solids.

DISCUSSION

GEORGE T. HAMMOND,* M. AM. SOC. C. E.—The behavior of Imhoff tanks, or indeed of any other variety of tanks used in sewage treatment, is a matter of much importance to sanitary engineers. It would be difficult to find any one more intimate with this subject than the author.

The principal functions of these tanks are to remove the suspended solids from sewage and to digest the solids removed. The extent to which the mechanical, physical, biological, and chemical agencies are concerned in the removal and reduction of suspended matter in sewage, constitutes a very extensive study, to which some of the ablest students have devoted much time and labor without exhausting its uncertainties.

In a previous discussion,† the author emphasized the fact that "the principal thing in which the sewage treatment plants in this country have not come up to a desired standard has been their operation. There have been rather few cases where design and construction, and even methods, have been such as to cause the plant to be justly termed a failure. There have, of course, been improvements from time to time. There will always continue to be improvements in methods and designs; but we seldom seem to secure satisfactory operation."

It is gratifying to learn that the plants described in this paper are carefully operated. This permits a fair comparison of results in tank performance such as is seldom possible.

The comparative dimensions, proportions, and cubic capacities of the sedimentation and digestion compartments are of much interest. It is obvious that every Imhoff tank has a limited capacity and that almost any tank might be expected to operate without nuisance within its capacity. This, however, is not as simple as it seems. What is tank capacity, and how may it be predicted? Unfortunately, tank capacity is not by any means a constant value throughout the year, and to answer such a question is not easy in advance of the installation, as there are many factors of doubtful value in the problem.

Under the circumstances, it would be wise to instruct the operator to find the capacity by test, regardless of any statement that may have been made to him, and, having satisfied himself of its capacity under varying conditions, to operate it within the limits of safety at all times.

Designers have not always been able to forecast the tank capacity correctly —it would be remarkable if they could—and no Imhoff tank will operate with success if overloaded. As an instance of actual and expected capacity, the speaker recalls that the Imhoff tanks at Baltimore, Md., soon after they were put into service, operating on about one-half the flow for which they were intended, appeared to be doing fairly good work, with no foaming; later, after the full load was applied, they foamed excessively and developed other troubles not observed in the previous instance. Under half load, the capacity of the digestive compartments was about 2.0 cu. ft. per capita.

* Cons. Engr., Brooklyn, N. Y.

† *Proceedings*, Am. Soc. for Municipal Impvt., 1922, p. 202.

The best way to manage a tank that proves to be too small is to work it only at its actual safe capacity, unless its design is so bad that it seems to have no safe capacity. The speaker has never seen a tank that would not operate with some success if the load was correctly determined and never exceeded.

The first requirement of an Imhoff tank is sufficient capacity for the greatest burden it may be called upon to carry. After this—and also of prime importance—are the details of design, and the correct proportioning of the parts. In every instance this should be considered in relation to the local conditions and service required, as well as to the prevailing climate, temperature changes, character of sewage, etc. If special studies of the local sewage are not made, as to volume of deposit, water content of sludge, etc., it will not be possible to forecast the probable tank capacity before the tank is in operation.

The author has covered details of design as related to the problems presented in his paper about as fully as such a study requires.

Concerning design, there is little or no criticism called for at this time, although faults in design may intensify other troubles in a tank. The designs exemplified in the Rochester and Fitchburg plants are generally accepted as standard by American engineers. The Plainfield installation has supplied experience that has been fruitful. The Schenectady plant was not built as designed.

The important difference in these plants is to be found in the comparative depths of the tanks, and in the cubic capacity per capita provided in the digestion compartments. Other things being equal, the speaker would expect that the tank having the greatest ratio of depth to capacity would give the most successful performance.

The author's treatment of the rate of sludge digestion and the per capita sludge accumulation in Imhoff tanks at relatively low and high temperatures, is very suggestive of the causes that lead to the overloading of a digestion compartment. The increase in required capacity per capita from 1.83 to 2.04 cu. ft. with a decrease of only 5° in temperature, is significant as to what may be expected to happen if ample space is not provided in this compartment.

It is probably a safe presumption that temperature effects take place more slowly in a deep tank than in a shallow one of equal capacity, and are not so marked; and that in shallow tanks, such as those at Schenectady, the maximum effects of temperature changes are to be expected. Actual, rather than average, temperatures are of importance, as in shallow tanks these changes may occur rapidly, slowing down the biological processes by lowering, and increasing them by raising, the temperature. These changes are doubtless more rapid in tanks that have a large surface exposure in relation to depth. Where the depth is relatively shallow, the sedimentation compartment reaches relatively deeper into the digestion compartment, and, therefore, transmits the temperature of its contents more quickly to the digesting sludge.

This view of the subject may explain the causes of foaming, especially in a shallow tank, a rapid increase in temperature causing a stimulation of the biological processes and a more rapid evolution of gases.

The importance of temperature studies in connection with tank performance is no doubt very great. It seems probable that a low temperature, in the cool months of the year, followed by a short warm season, may result in an abnormally large accumulation of digestible solids while the temperature is unusually low, followed by a violently active digestion of sludge in the warm period. Winter foaming is seldom observed.

The phenomena which accompany foaming and frothing in Imhoff tanks are, however, so divergent that it must be admitted that many explanations are possible. Tanks will foam when no sludge is present; or when filled with sludge; or when much or little sludge is present. The suspended matters are so light in specific gravity, that any causes which give rise to a rapid gas formation may result in foaming, such as a sudden increase in temperature, or even a change in barometric pressure. The presence of grease in the tank and an acid reaction are conditions often noted when foaming occurs. Foaming, however, takes place when the condition is not acid and there is but little grease present, and when all explanations seem insufficient. At times during the Brooklyn experiments, and without any apparent reason, the entire body of sludge in the digestion compartments of the different tanks would rise to the top, usually without foaming or frothing, remain for a day or two, and then slowly settle. On such occasions, gas discharge from the gas vents was at least of more than average activity and eructations of gas were noted. At such times there was considerable odor present, and when foaming occurred there was a very persistent odor. Churning the scum with a hoe in the gas vents, or breaking it up with a paddle, usually sufficed to release the entrained gas and permit the mass to settle. At no time was the foaming so serious that it could not be controlled, but while it lasted the smell was obnoxious. The sedimentation compartments were at no time involved and very seldom showed any scum. The "free-board" was only 15 in. and was never overtopped.

It may be added that the Imhoff tanks at Brooklyn, N. Y., seldom caused odors, and when they did, if it was not due to foaming—noted only on very warm days in summer—it was probably caused by a septic condition of the sewage entering them. At one time during the experiments all the tanks emitted odors, and it was discovered that the main sewer had become very foul from deposits. After the sewer was cleaned, these odors ceased.

The idea that sewers and disposal plants will run without causing trouble if left to themselves is erroneous as all know; but the benefit from cleaning the sewers is not sufficiently recognized.

An excessive formation of scum is usually not observed in sewage from a combined system, although frequently in sewage that is relatively fresh, especially if from a separate system. This problem at times constitutes one of the greatest difficulties of the plant operation and is often related to foaming. Most frequently the scum consists of unbroken sewage matter—paper, sticks, fresh vegetable remains, shreds of string, trade wastes, rags, hair, etc., and a considerable quantity of fatty material—all these constituents being intermingled and in a condition that resists rapid bacterial action. The various scraps from the making of home brews are strongly in evidence, and very resistant to bacterial attack.

The effect of scum is, first, mechanical, as it plugs up the gas vents and even reaches below the slot line. Its influence on the bio-chemical processes of the digestion compartment has not been sufficiently studied, but it would appear at times to be the cause of an acid condition. Under all circumstances it is a nuisance to the operator.

Where the sewage produces large quantities of scum, as at Plainfield, fine screens would afford the best possible assistance in the operation of the plant. The occurrence of excessive scum was not noted at Plainfield previous to the installation of the Imhoff tank. There were at first four septic tanks—two, 100 by 50 ft., built in 1902, and two, 200 by 50 ft., built in 1905. These tanks were covered and had a working depth of 6 ft. They could be operated in parallel or in series. In 1908, they were reported "to be working well, and there is not a very heavy scum on the surface."* These tanks were succeeded by the existing plant, which serves a population of about 40 000, in the three municipalities of Plainfield, North Plainfield, and Dunellen, N. J.

The Plainfield Imhoff plant is one of the earlier plants in this country, and some of the theories under which it was designed have become modified by experience there and elsewhere. The digestion compartment is divided into five chambers. The greater part of the sedimentation took place in the chamber under the inlet for the sewage, and three of the remaining chambers accumulated little sludge. As a remedy, the rate of flow was greatly increased, so that all the chambers might be useful, but this led to troublesome scum formation and foaming. Next, a fine screen was used, and this considerably reduced the formation of scum, and with some other changes greatly improved the operation.

If at first all of the sludge digestion chambers had been combined into one compartment throughout the length of the tank, it seems probable that much of the trouble would have been avoided. The depth of the digestion compartment and the length of the sedimentation compartment should prove sufficient for good operation, although the depth might have been greater with advantage and the sludge digestion capacity is apparently deficient.

The Schenectady plant is notable for its shallow depth—on account of construction difficulties, the original design, 21.33 ft. deep, was changed to 13.42 ft. It was put into operation in January, and the sludge had not ripened when the first withdrawal was made in the following June, so that it was very offensive. The addition of lime was apparently of little use; foaming began and persisted in flowing over the wall of the gas vents into the sedimentation compartment.

It is to be noted that the raising of the walls of the gas vents and the free interconnection of the chambers constituting the digestion compartment have been effective in greatly improving the conditions. With the occasional use of lime and with other expedients the tank now works.

In this case the faults of the Plainfield plant appear to be intensified. Its failure to function seems to have arisen from much the same causes, with the added difficulty of very shallow depth. It is possible that this also would have been a favorable condition for the fine screening of the influent.

* Thirty-second Annual Rept., New Jersey State Board of Health, 1908, p. 408.

In contrast the operation of the Rochester and Fitchburg tanks is strikingly different; yet in these latter plants functional disturbance may be observed and even occasional foaming (at Fitchburg). These difficulties appear to be caused chiefly by (1) the changing temperature of the sewage, and (2) its effects on the bio-chemical activity in the various compartments.

Unfortunately, so little is known at present of what actually takes place in the various compartments of an Imhoff tank that any picture one may draw is wholly inadequate and largely futile.

From an examination of the four plants described in the paper the following comments seem obvious:

(*a*) Depth of tank is an advantage. It not only improves the storage conditions and the concentration of ripe sludge at the sludge pipe outlet, but it also makes for more uniform temperature and slower changes, especially in the contents of the digestion compartment.

(*b*) Shallow tanks cover a relatively larger area, and are more subject to rapid changes in temperature. Sludge storage conditions in them are not so favorable as in deeper tanks; and the relatively larger area of the top surface, even if partly covered, affords a greater opportunity for scum formation, and quite possibly for acid conditions derived from the scum.

(*c*) It is apparent, and quite in line with the opinion of many sanitary engineers, that there should be not less than 2 cu. ft. of effective capacity per capita in the digestion compartment. Indeed, the tendency to increase this provision appears to be logical. Ample capacity affords an assurance of safety in operation.

(*d*) The removal of the finer, or colloidal, matters in suspension is advantageous, and may be accomplished by detention periods that, although relatively long, are not accompanied by any bad effects in a well proportioned tank; but detention for periods longer than three hours may be unfavorable. In some tanks, a shorter period than three hours is advisable.

John F. Skinner,* M. Am. Soc. C. E.—The author has brought to the consideration of this subject so much thought, experience, wisdom, and imagination, that he seems to have touched on all the reasons suggested for the difficulties in the operation of Imhoff tanks. His selection of four well known plants that have been visited by many sanitary engineers and municipal officials, and represent such different conditions, has been very happy even if the presence of so many variables makes a solution somewhat indeterminate.

In September, 1912, the speaker was privileged to take part in a two-day conference with Dr. Imhoff, the late Emil Kuichling, M. Am. Soc. C. E., and Edwin A. Fisher, M. Am. Soc. C. E., then City Engineer of Rochester, N. Y., at which time the difference between conditions in the Emscher District, Germany, and in northern American cities was most intimately discussed, with especial reference to Rochester.

Since then, the speaker has designed three more plants with Imhoff tanks, that have been constructed at Rochester, namely, the Brighton, Charlotte, and University Plants. The Irondequoit, or main Rochester Plant, referred to

* Cons. Engr.; Deputy City Engr., Rochester, N. Y.

by the author was designed by the late Mr. Kuichling; to his forethought is chiefly due its success, although the half of the plant first built was designed for a population of only 200 000 and is now operating with a 30% overload.

This discussion will be devoted mainly to the Rochester plants with such comparisons as suggest themselves and general conclusions therefrom.

Character of Sewage

All the Rochester plants receive typical domestic sewage with no preponderating trade wastes. The water, as stated by the author, is slightly softer than that at Plainfield, N. J.

The Irondequoit Plant receives combined sewage, the Charlotte Plant also receives some storm water. The Brighton Plant serves a sanitary district only, and the University Plant (just constructed) will receive sanitary and hospital sewage.

Freshness of Sewage.—The raw sewage received at the Irondequoit Plant is stale and often in a septic condition when it reaches the plant. Sewage reaches the Brighton Plant within an hour. There is more colloidal matter in the Charlotte sewage than at the other plants, and more odor, possibly due to the presence of the products of solids that have deposited during dry weather in some of the larger combined sewers which are still connected with the system.

Hardness of Water.—The Rochester and Lake Ontario Water Company supplies water to the Brighton and Charlotte territories, its hardness being 122 parts per million as compared with 65 for the Hemlock System which supplies most of the city.

Both kinds of water are supplied to the two Rochester plants (Irondequoit and Charlotte) that develop quantities of scum. The sewage of one is fresh and that of the other stale; the only common factor is that they both receive street wash, which seems to be the source of most of the scum. The hardness of the water has an effect, however, in that grease balls are found in the scum at Brighton and at Charlotte, as at Irondequoit. The action of grease will be discussed subsequently.

Preliminary Treatment

All sewage entering the Irondequoit Plant passes through racks with 3-in. clear openings, then through detritus tanks, 90 ft. long, at a velocity of about 1 ft. per sec., and then through Reinsch-Würl screens with $\frac{1}{8}$-in. by 2-in. slots. By the time the screen plates are discarded the slots have been worn somewhat wider.

At Brighton, grit-chambers are provided for emergency, followed by flat, inclined, fine racks with $\frac{1}{4}$-in. clear openings.

At Charlotte, there are detritus tanks 40 ft. long, in which a velocity of about 1 ft. per sec. is maintained in shallow channels with V-shaped bottoms. Fine racks are used similar to those at Brighton, but more nearly vertical in position.

With sewage as old as at Irondequoit, fine screens are useful, as the material removed is mainly garbage and other relatively non-putrescible substances.

The screenings would add 4 to 6 cu. ft. per 1 000 000 gal. of relatively inert matter to the contents, considerable of which would have to be skimmed from the settling chambers. Feces are largely comminuted before reaching the plant and pass through the screens. At Baltimore, Md., on the contrary, with equally old sewage, great masses of unbroken feces are raked from the racks. This is one of the many features which indicate that each situation must be studied individually in the preparation of a satisfactory design. With the fresh sewage at Brighton and Charlotte, feces are not so well comminuted. The fine racks collect rags, fruit rinds, and paper pulp, while feces are broken up at intervals with a hose stream and sent on to the tanks.

Subsequent Treatment

The designer of a tank must consider the subsequent disposition of the effluent. The Irondequoit Plant discharges into Lake Ontario 7 000 ft. from shore in water 50 ft. deep. The winds, currents, and temperatures of the lake and sewage are such that there is no shore contamination. A careful preliminary study of the conditions was made by the late George C. Whipple, M. Am. Soc. C. E., in 1913, and his work was repeated by Mr. M. J. Blew in 1921, after the plant had been functioning for four years. The findings of Professor Whipple were confirmed in every respect. The design of the plant called for a detention period of 80 min.

The Charlotte Plant discharges into the non-potable water of the Genesee River where ample dilution is provided. As the population is so variable, unusual conditions had to be met in the design, namely, the operation of the plant at a minimum discharge of 30 cu. ft. per min. and a maximum of 180. The territory includes a summer colony, a municipal park, and a bathing beach having a transient population. The detention period, therefore, is from 1 to 6 hours.

The effluent at the Brighton Tank, like that at Schenectady, N. Y., Plainfield, and Fitchburg, Mass., is sprayed on trickling filters. A detention period of at least 2 hours was provided for; it is about 3 hours at present.

The University Plant is similar in design to the Charlotte Plant, only smaller. It will discharge into the non-potable waters of the Barge Canal. It is constructed as a temporary plant and will serve a territory of 300 acres, including the Medical School of the University of Rochester now under construction. At some future time, as the city develops, this section will become a part of a larger territory tributary to a plant to be constructed about five miles to the east.

Typical cross-sections of the tanks listed in Table 16 are shown in Fig. 4.

Foaming

Foaming has never occurred at the Brighton Plant or at the Charlotte Plant and only once, five years ago, in one of the ten Irondequoit Tanks. This was immediately corrected by drawing sludge. It was distinctly a matter of improper operation and occurred when the man in responsible charge was serving in the Army.

Scum

Considerable scum forms at the Charlotte and Irondequoit Plants, consisting of leaves, matches, rubber, and grease balls. It is usually inoffensive and is removed when it reaches a depth of 1 ft. or more. Occasionally, sludge appears in the scum; this is hosed and paddled and thus driven down. Gas action is still (January, 1924), noticeable at the plants, as the season has been quite warm. The presence in the scum of matches, which are not often seen floating in the settling chamber, has caused speculation. It has been suggested that they come into the tank smeared and weighted down with other material and float after being disentangled; on the other hand, they may have been water-logged and later floated by the formation of gas within them.

TABLE 16.—Characteristics of Imhoff Tanks at Rochester, N. Y.

Plant.	Irondequoit.	Brighton.	Charlotte.	University.
Designed for population of	200 000	15 000	10 000	3 000
Tributary population	260 000	10 000	4 000	0
Character of Sewage:				
Flow, in million gallons daily	32	1.2	0.33	0
Separate or combined	Combined	Separate	Combined	Separate
Suspended solids, in parts per million	163	106	390	.....
Freshness	Stale	Fresh	Fresh	Fresh
Industrial wastes	Small amount	Small amount	Small amount	Hospital
Hardness of water supply, in parts per million	65	122	122	65
Temperature, in degrees Fahrenheit	54	53	54	.....
Preliminary Treatment:				
Screening	Fine screens	Fine racks	Fine racks	Fine racks
Number of units	10	1	1	1
Grit chambers	Yes	Yes	Yes	None
Skimming tank	No	Yes	Yes	Yes
Sedimentation Tanks:				
Sedimentation period, in hours	1.1	3	6	2
Total depth of tanks	33 ft. 10 in.	34 ft.	30 ft. 6 in.	25 ft. 2 in.
Depth to slots	11 ft. 4 in.	{ 13 ft. 8 in. / 19 ft. 0 in. }	15 ft. 2 in.	9 ft. 8 in.
Effective length of flow chambers	110 ft.	60 ft.	42 ft. 9 in	26 ft. 0 in.
Width of flow chamber	10 ft.	24 ft.	13 ft. 9 in.	9 ft. 6 in.
Number of flow chambers	2	1	2	2
Sludge Compartment:				
Depth below plane of slots	22 ft. 6 in.	15 ft. 0 in.	15 ft. 4 in.	15 ft. 6 in.
Depth below 18-in. neutral zone	21 ft. 0 in.	13 ft. 6 in.	13 ft. 10 in.	14 ft. 0 in.
Depth of sludge hopper	8 ft. 0 in.	5 ft.	5 ft. 6 in.	5 ft. 6 in.
Cubic feet per capita	2.4	2.18	2.88	1.47
Number of hoppers	3	2	1	1
Ease of communication	Good	None	.....	.....
Solids, in cubic feet per capita per year	0.321	0.236	0.214	.....
Gas Vents:				
Percentage of tank area	26.8	12.9	6.75	5.12
Area, in square feet per cubic foot of sludge per capita	0.0153	0.0085	0.0061	0.0064
Least width of vent	2 ft. 6 in.	2 ft. 2 in.	9 ft. 6 in.	6 ft. 0 in.
Horizontal travel of bubbles	0 to 6 in.	0 to 12 ft. 10 in.	0 to 13 ft. 3 in	0 to 9 ft.
Date first operated	March, 1917.	March 1, 1916	October. 1921	.....

Little scum has been observed at the Brighton Plant and then only in detached masses, although the gas action is very free. This tank is unique in design in that it has only one settling chamber with four bottom slots, two gas vents, and two separate sludge compartments.

Sludge

The sludge from all the plants is well digested and inoffensive. It does not attract flies at any time and readily dries on the beds. It has contained as little as 79% of moisture and as much as 95%; it is dryest at the first drawing in the spring and less dense later. There is sufficient space in the tanks so that a drawing of well digested sludge can always be made at the advent of drying weather in the spring.

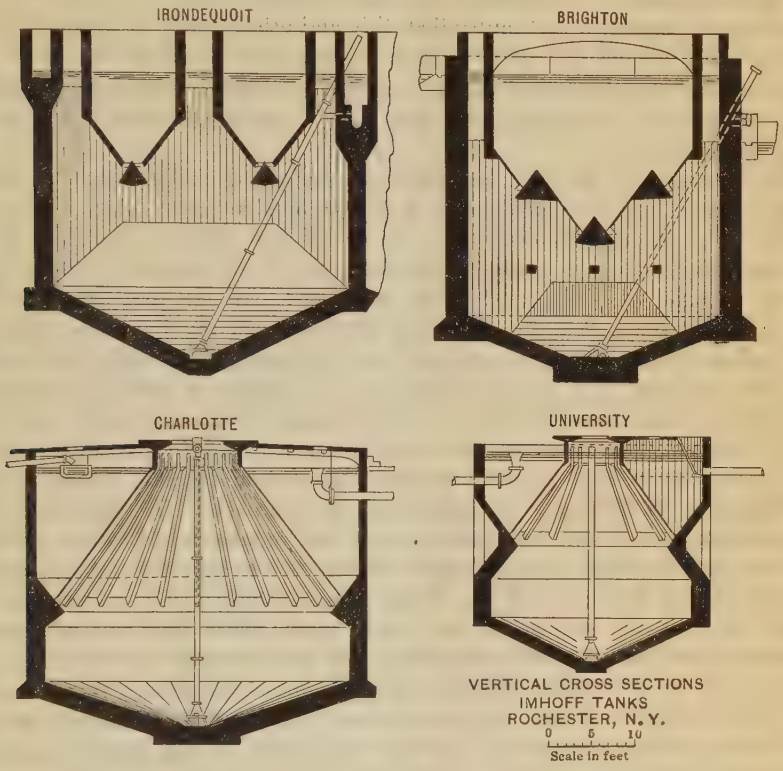


Fig. 4.

Grease

Insoluble soaps, formed by the substitution of calcium and magnesium in hard waters for the more loosely combined potassium and sodium in soaps, have been suggested as formers of scum. As found at Rochester, these grease balls are pellets 1 to 10 mm. in diameter. They do not coalesce and, therefore, do not cause foaming. Glycerine and saponifiable oils in solution will doubtless increase the surface tension in the bubbles and hence the tendency to foam. At the same time, emulsified oils may coat particles of organic matter and prevent complete digestion, or may coat them so that the gas forming within them will not be liberated readily and will thus cause the particle to rise and remain in the scum.

Physical Action in the Digestion Chamber

If there is a mass of sludge in the bottom of a tank there will probably be a newly deposited, loosely bound, fecal mass, mainly organic, with grains of mineral matter entangled in it near the upper surface of the previously deposited sludge. When a bubble of gas, released from below, rises through this loose mass its path may fill with water leaving a minute channel, roughly vertical, into which a particle of mineral matter, disentangled by the rupture due to the progress of the bubble, may drop a short distance. As this operation is continued, repeated, and multiplied, this and other grains of mineral matter will be disentangled and drop through the mass of sludge to the lower part of the sludge chamber, resulting in a greater proportion of mineral matter at the bottom in the sludge as drawn than in the suspended solids of the sewage or in the scum.

At Irondequoit, in 1922, the ash from suspended solids from the sewage averaged 28% and from the sludge as drawn 56.4 per cent. Ash from solids in the scum averaged 26.6 per cent. From the first drawing of sludge in the spring of 1922, grit was recovered by elutriation to the amount of 13.7% (dry basis). Again, on July 7, 3.8% was recovered. In the first sample, all of which passed through a 20-mesh sieve, 20.8% passed the 100-mesh sieve. In the second sample, 11.6% was retained on a 10-mesh sieve, while 7.7% passed the 100-mesh and 1.9% the 200-mesh sieve. A sample of scum was examined and after elutriation and decantation showed only 0.5% of grit (dry basis).

In general, therefore, the free mineral matter and the inorganic matter are greatest at the bottom of the sludge chamber and decrease toward the top.

The effect of difference in temperature is noticeable. In November, 1922, the temperature of the Irondequoit sewage had reduced to 54° Fahr., while it was 56° in the body of the sludge. When the temperature of the sewage becomes lower than that of the sludge, digestive activity is checked in the cooler zone at the surface of the sludge mass where light matter—mostly organic—is deposited. This material may lie relatively quiescent on the surface, for there is little mineral matter from street wash at this season of the year to weight it down. On the return of warm weather, this finely divided and readily digested material, agitated by rising bubbles from below, will digest with increasing rapidity and, if the surface of the sludge is too high, will show itself in the slots at the bottom of the settling chamber, and, after these are sealed, by rising in the gas vents. This sequence of events can be averted to some extent by agitating the sludge during the winter so that the freshly deposited suspended solids may be distributed throughout the sludge mass and thus be partly digested during the cold period. This will have two beneficial effects—the reduction of quantity and the spring level of the sludge due to liquefaction and gassification, and the diminution of the violence of digestive action near the top of the sludge mass in the early summer.

Causes of Foaming

Consider the phenomenon of foaming from the standpoint of an air lift. As gas is liberated and rises through the sewage in the scum compartment (the

slot being free), the condition to produce overflow of the vent would be that the higher column in the scum and gas chamber must equal in weight a column of sewage in the settling chamber. In other words, if h_1 represents the depth of sewage to the slot and h_2 the free-board of the gas-vent walls, the average density of the rising liquid will be $\frac{h_1}{(h_1 + h_2)}$ and the ratio of gas bubbles to volume in the scum and gas chamber will be $\frac{h_2}{(h_1 + h_2)}$. This condition can hardly be imagined; it would amount to $\frac{1.5}{(11 + 1.5)} = 12\%$ of gas at Plainfield; 11% at Schenectady; and 8.4% at Fitchburg.

If, however, the slot is sealed by the rising sludge not only may a condition exist more favorable to foaming from the standpoint of balanced columns but, as the rising sludge level contracts the horizontal area of the scum chamber, the gas will rise through a smaller quantity of liquid, more readily approaching air-lift conditions.

If, in addition, a mass of scum is added, with the surface tension of the bubbles increased by the presence of saponified material, and, further, if the gas vent is so narrow that scum and bubbles coalesce and bridge across the opening without breaking, conditions are ripe for foaming.

In the speaker's opinion, the causes of foaming at the various plants considered by the author are, as follows:

Schenectady.—Design.—

(*a*) Shallow tanks, eight in line, may be only about three-eighths efficient in digestion and release of gas, hence the loading may be eight-thirds as great as indicated by actual capacities.

(*b*) Small sludge compartments made smaller in effect by poor intercommunication.

(*c*) Trough bottoms do not lend themselves as well to sludge drawing as hoppers.

(*d*) Narrow gas vents.

Plainfield.—Design.—

(*a*) Five tanks in line may be only three-fifths efficient in digestion and release of gas, hence the loading may be five-thirds as great as that indicated by actual capacities.

(*b*) Small sludge compartments made smaller in effect by poor intercommunication.

(*c*) Narrow gas vents.

Fitchburg.—Design.—

(*a*) Tall, narrow, scum compartments and gas vents favor air-lift action.

Fitchburg.—Operation.—

(*a*) Sludge carried too high.

Rochester.—Operation.—

(*a*) Sludge carried too high.

Determination of Sludge Capacity.

The author suggests that "cubic feet per capita" is not the soundest basis for designing the capacity of an Imhoff sludge compartment. This, again, is another way of saying that each situation must be studied individually.

Attention is called, however, to the comparison of the four plants shown in Table 17.

TABLE 17.—Theoretical Comparison of Sludge Accumulation.

Plant.	Sewage, flow, in gallons per capita per day.	Suspended Solids.		Sludge, 90% water, in cubic feet per capita per month.	4 Months' Collection, in Cubic Feet per Capita.		Quantity after 25% Settlement and Digestion, in Cubic Feet.		Dr. Imhoff's rule, in cubic feet per capita.
		Parts per million.	Cubic feet per capita per month.		50% removal.	75% removal.	50% removal.	75% removal.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Schenectady	92	163	0.060	0.60	1.20	1.80	0.90	1.35	.....
Plainfield	85	166	0.056	0.56	1.12	1.68	0.84	1.26	.....
Mean, Sanitary	.....	.....	0.058	0.58	1.16	1.74	0.87	1.30	1.20
Fitchburg	89	219	0.078	0.78	1.56	2.34	1.17	1.76	.....
Rochester	123	163	0.080	0.80	1.60	2.40	1.20	1.80	.....
Mean, Combined	.....	.....	0.079	0.79	1.58	2.37	1.185	1.78	1.80

It is obvious that by whatever method the proper capacity for the sludge chamber is found, it is directly proportional to the population and therefore, ultimately, must be on a per capita basis. It is equally obvious that it is directly proportional to the quantity of sewage per head per day, to the suspended solids and the proportion of them removed by sedimentation, and to the number of days in the period when sludge cannot be drawn, due to weather conditions. It is also inversely proportional to the solid content of the sludge and is reduced by the amount of digestion during the cold period.

In determining the capacities of various parts of a tank, one should first, for the particular set of conditions, design the sedimentation chamber for a desired percentage removal of suspended solids. Having assumed this quantity and the total per capita suspended solids in the sewage and the length of period during which sludge cannot be drawn from the tank, a general formula may be evolved, as follows, assuming sludge with 90% of water in the tank, and 25% reduction by settlement and digestion during the cold period:

$$Q = \frac{D\,G\,R\,S \times 10}{1\,000\,000 \times 7.5\ (\text{gal. per cu. ft.})} \times 0.75 = \frac{D\,G\,R\,S}{1\,000\,000}$$

in which

D = Period, in days, when sludge cannot be drawn;
G = Gallons of sewage per head per day;
R = Portion of suspended solids removed, expressed decimally;
S = Suspended solids in sewage in parts per million;
Q = The sludge space required, in cubic feet per capita.

In Table 17, Column (5), there is an assumption which may be varied to suit given conditions of temperature, etc., affecting the rapidity of digestion. The same is true of the 25% reduction assumed in Columns (8) and (9).

This equation is submitted as a convenient and flexible method of reaching a reasonable solution of an important problem. Like all determinations involving variables of unknown definite value, a reasonable factor of safety is to be commended as well as a careful comparison with existing plants.

Aids to Proper Functioning

It is evident that the most careful operation cannot prevent foaming where the design is at fault or where the plant is overloaded, likewise that careless operation may result in difficulties with the best designed plant. It would seem that the prime requisite is a liberal and well balanced design and one which considers ease of operation.

The following points are suggested as important:

Design.—

(*a*) Gas vents: Wide, but not necessarily of large area.

(*b*) Scum chambers: Ample in width and connecting freely with the gas vents.

(*c*) Sludge compartments: Ample in capacity and not more than three in line, in which case the intercommunication must be easy. Ample means of agitating the sludge with water jets, should be furnished.

(*d*) Settling chambers: Design with relatively large area and shallow overflow, and with provision for reversal of flow in the case of more than one sludge compartment.

(*e*) Preliminary treatment: Provide fine screens or fine racks and skimming chambers or deep scum-boards, so that large objects, floating matter, and scum may be kept out of the tank.

Operation.—

(*a*) Routine: Keep screens cleaned and floating matter skimmed. It is advisable to drive down undigested material in the gas vents and to remove the digested scum when it reaches 1 ft. in thickness. Agitation of the sludge some time before drawing and during the winter, if the sludge capacity is not large, will prove effective. A regular reversal of the flow—fortnightly or monthly—depending on the relative depths of sludge in the different compartments, is essential.

(*b*) Records: Keep records of temperatures and quantities and all operations both physical and laboratory, also, a journal, recording those conditions and occurrences not readily tabulated.

JOHN R. DOWNES,* ESQ.—The author is to be sincerely thanked for bringing together the available information on Imhoff tanks and focusing it on four installations of known operating characteristics.

The speaker approves Mr. Eddy's statement as to the two primary functions of Imhoff tanks, namely, the deposition of solids in the upper compartment and their digestion in the lower compartment. Many persons seem unable to rid their minds of the idea that the passage through the tank does something to the liquid other than to separate it from the solids. Engineers should disabuse their minds of this relic of septic tank lore. The experience at Plainfield, N. J., checks nearly all the author's conclusions except as follows:

Although it is strictly true that the degree of removal of suspended solids depends primarily on the length of the detention period (with due attention to velocities) this has little application within practical limits in the case of the Plainfield sewage. In 1912, Weston Gavett, formerly Acting Chemist in Charge at the old Plainfield plant, showed that practically all sedimentation occurred within 50 to 60 min. after entrance. Although his work was with quiescent settling in the laboratory, it practically checks the actual experiences of operation.

The speaker disagrees as to the digestion of the minute solids and believes it would be better to state that the more finely divided solids which are removed to a greater degree by a long detention period are in a state of division more accessible to the disintegrating organisms, but the success of the attack depends on the composition of the solids in question. At Plainfield, these finely divided solids are largely composed of grease and are digested with the greatest difficulty. This point is covered in a way by the statement as to the effect of excess soap grease at Plainfield and at Schenectady, N. Y.

The author states that at Schenectady it was possible by building up the walls of the gas vents to cause the foam to pass over into the sludge pipe. At Plainfield, this was not successful as the foam broke in passing over the edge, drying instantly and refusing to run down the pipe.

An imitation of the dense, ripe, quick-drying sludge produced naturally at Rochester, N. Y., and Fitchburg, Mass., can be produced from the thin, watery sludges obtained at Plainfield and Schenectady by the addition of a small quantity of alum. Observation of the transition of the sludge from the lighter to the denser state suggests that time alone would accomplish the change if the size of the digestion chamber were great enough to permit digestion to proceed to its ultimate end. Whether this change is due to the continued action of the same groups of organisms, to the substitution of other organisms for them, or to the reaction of the end products of one group on those of the other group, is not now clear. Either of the latter two views would help to explain the necessity for the maintenance of a preponderance of the ripe sludge in the tank in order to keep control of the digestive process. The difference in capacity of the digestion chambers at Rochester and Fitchburg on

* Superv. Engr., Plainfield, North Plainfield and Dunellen, N. J., Joint Sewage Disposal Works, Bound Brook, N. J.

the one hand and at Schenectady and Plainfield on the other, is sufficient to allow for this time factor and, therefore, to explain largely the difference in action.

The author states that there are approximately 55 grammes per capita of suspended solids at the two plants treating domestic sewage and 75 grammes per capita at the two handling combined sewage. The extra 20 grammes, which is ascribed to street wash and is probably of a gritty, mineral, nature, may well be of actual service as a dilutent of the slimy, greasy mass of solids. A small proportion of sand in the solids may render them more penetrable to the attacking organisms, just as a small portion of sand in a dense clay will render it more permeable to water. The slight migration of such mineral solids due to gas movement in the mass may even produce an attrition effect. The solids from street washings in this day of paved, horseless streets must be more of a mineral nature and less of a burden on digestion capacity than was the case when the conception of the effect of such solids was established twenty years or more ago. Such solids, although represented by a weight of 20 grammes in 75, comprise only about 1% by volume of the freshly settled solids.

In computing Table 8 the author assumes that the sludge will contain 15% of solids at the bottom of the tank, and 10% at the surface. It is necessary to assume some such figure as a starting point; but this is the very assumption which seems to have created all the disturbance at the Plainfield plant. The designer has told the speaker that he calculated the sludge storage capacity on the basis of the results obtained at Fitchburg where sludge containing 12 to 15% of solids is common. The author has pointed out, on the other hand, that the conditions at Plainfield result in a sludge of about 7.5% of solids; actually, 6% would be more nearly the average.

The speaker agrees with all the author's six points stated in Conclusions 25 (*a*) to (*f*), as to the essentially unfavorable conditions at Plainfield and Schenectady.

The New Jersey Sewage Experiment Station has under way several of the investigations mentioned by the author, including temperature, hydrogen-ion concentration, quantity and composition of gases, and kind and function of organisms. It is not possible to study tanks of different depths at Plainfield, but different depths of sludge compartment will be compared.

The first trouble was due to acid-foaming with attendant odors. The later foaming has been alkaline, with little odor. Foaming always occurs when the material is near neutral or is acid. It was a question whether there was a lack of sludge digestion space or of sludge drying space. The use of alum solved this problem, by hastening the time of drying of the sludge so as to permit its removal from the tanks as fast as it was ready. The trouble is that there is not enough space to maintain the necessary preponderance of old sludge and thus control the digestion.

The difficulties encountered at Plainfield have been:

1.—The excessive formation of scum resulting in (*a*) odors, and (*b*) excessive manual labor to remove it.

2.—Return of solids through the slots to the sedimentation compartment and the consequent fouling of the liquid; and, again, an excessive labor demand for cleaning the surface.

All these difficulties may be said to be due to the fact that the solids neither stay at the bottom to form sludge nor rise to the surface to release the entrained gases and re-settle by gravity. A considerable portion of the solids has a specific gravity less than unity, being of a fatty nature, and, after being released from the entangling heavier solids which originally dragged them down into the lower compartment, remains permanently at the surface. Worse than this, however, the gases which buoy up the remainder of the solids are in the form of minute bubbles, mere pin-points, such as give the milky appearance to air-filled tap water. True, the minute bubbles escape from the tap water in time; but in the Imhoff tank the buoyancy of these small bubbles is exactly counterbalanced by the weight of the solids in which they are entrained and held as individual bubbles, so that gases and solids hang suspended together in an almost jelly-like mixture with the liquid, filling every part of the digestion compartment.

In other plants where the gas forms in bubbles of sufficient size to carry solids to the surface with some momentum, there may be a neutral zone about the slot and the trapped slot becomes effective in preventing the return of solids by diverting the course of the rising gas and solids. Where the mixture hangs suspended, however, as at Plainfield, there is no momentum; consequently, there can be no diversion, and the trapped slot becomes useless. The solids thus suspended may amount at times to as much as 3 per cent. Interchange between the compartments by diffusion, thermal currents, and displacement cannot be prevented.

At first glance the difference between the cross-sections of the Plainfield and Fitchburg plants seems to lie mainly in the communicating opening between the hoppers. A study of the different volumes resulting from the two kinds of hopper treatment is interesting. The author states that the difference in depth below the slot is 31% (Fitchburg, 9.5 ft., Plainfield, 7.25 ft.). The effect of the Fitchburg hopper, however, is to concentrate the heavier, older sludge from an area of 900 sq. ft. in approximately the same area as that filled from 150 sq. ft. in the Plainfield hopper. In Fig. 5 one hopper is shown superimposed on the other. One large Fitchburg hopper corresponds to four Plainfield hoppers and covers six times the area. The shaded areas in the apex of the hopper show the position which would be occupied by the contents of 1, 2, 3, and 4 Plainfield hoppers. The result is a concentration of four volumes of the best sludge on one unit area and the consequent increase of gas evolvement within this unit area. The effect of evolvement of gas from sludge concentrated on a comparatively small area, as compared with evolvement over a scattered area, is indicated in Fig. 6. To the right the minute pin-point gas bubbles are shown to be held individually by the entangling solids; at the left the four small bubbles occupying the space formerly occupied by one bubble must necessarily be nearer each other, and so will collide to form large bubbles. These bubbles, having sufficient buoyancy to rise with consid-

erable momentum, are, with their burden of solids, diverted by the trapped slot, whereas the few individual small bubbles, weighted down with solids, hang suspended and, having no momentum, cannot be deflected from the slot. Whether this effect of concentration of sludge and gas is accompanied by lowering the bottom of the tank, by raising the slot, or by use of a few large hoppers replacing several small ones, comes to much the same thing.

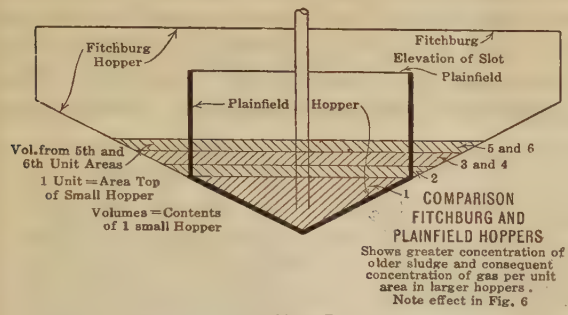

Fitchburg Hopper
Fitchburg Elevation of Slot
Plainfield
Plainfield Hopper
Vol. from 5th and 6th Unit Areas
1 Unit = Area Top of Small Hopper
Volumes = Contents of 1 small Hopper
5 and 6
3 and 4
2
1
COMPARISON
FITCHBURG AND
PLAINFIELD HOPPERS
Shows greater concentration of older sludge and consequent concentration of gas per unit area in larger hoppers. Note effect in Fig. 6

FIG. 5.

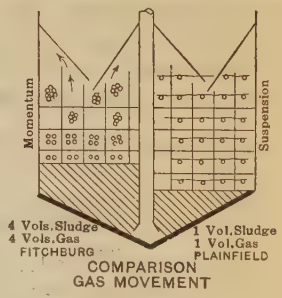


FIG. 6.

By good fortune Dr. Imhoff, the inventor of the tanks, spent an afternoon late in September, 1923, at the Plainfield plant. His reaction to the design and operation of the Plainfield tanks is of interest.

In anticipation of his visit, the superintendent had all the slopes, slots, and settling compartment surfaces cleaned with particular care 24 hours before his arrival and then gave orders that the surfaces were not to be touched again until Dr. Imhoff had seen them 24 hours later. The moment he came, he demanded, "What is that black mass doing on the settling compartment?". It was explained that this was the very question which he was expected to answer. Samples were then pumped from every part of the digestion compartment, showing that the condition previously described existed throughout. The sludge level was well below the neutral zone.

When Dr. Imhoff saw the plans, he stated that the sludge depth below the slot was far too limited. He was very emphatic in stating that good results could never be expected with such an arrangement—that it is useless to provide a given number of cubic feet of space without regard to the depth factor. As to the value of depth below the flow line, his answer was, "That is nothing, you must have depth below the slot". It is to be noted, however, that in his writings* he has laid stress on the value of the total depth.

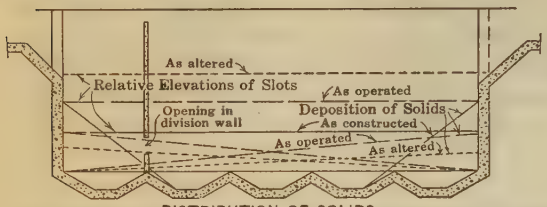
Another criticism which he made was that the opening below the curtain wall should have been closed. He stated:

"You cannot successfully work two tanks over one bottom; and that is what you are trying to do. However, if you raise the slot so as to get more depth below it, you may keep the opening submerged in the heavy ripe sludge, in which case it would not do much harm".

He also emphasized that the communicating opening between hoppers in the same row should be either lowered so as to be always submerged in the ripe sludge, or closed.

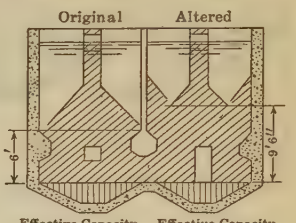
* "Eight Years of Imhoff Tank Design and Operation," *Engineering News*, Vol. 75 (1916), p. 14.

As to the operation—working only three tanks at one time—Dr. Imhoff pointed out that this plan was wasteful as a great and sensitive organization had been built up within the digestion chamber capable of handling a certain quantity of organic matter daily. During the period of intense feeding the organization attempts to build itself up to the work in hand and just as it is about established on this basis the supply of food is cut off and the vast organization goes hungry. He stated that it would be just as reasonable to try to feast two days a week and starve the remainder. However, when the reason for the plan of operation was pointed out, he agreed that this was the only course open under the local handicap of very poor distribution of the solids.



DISTRIBUTION OF SOLIDS
PLAINFIELD, N. J.
IMHOFF TANK

FIG. 7.



CAPACITY PLAINFIELD TANK
BEFORE AND AFTER ALTERATION
Compare Fig. 10

FIG. 8.

When all six tanks are used, the velocity of the settling compartment is so low, and in particular is reduced so suddenly, as the flow enters from the distributing channels, that practically all the solids simply drop into the first hopper, filling the digestion compartment to the slots in a few days. Even with only three tanks in operation, the deposition of solids in some cases amounts to 12 in. in depth per day, at a point, 2 or 3 ft. from the inlet end. This was determined by hanging buckets in the settling compartments. Contrary to the general belief, this deposit of solids does not compact rapidly, the first tendency toward mechanical compaction being counteracted by gas distention, as soon as disintegration sets in, so that solids deposited to such depths may not shrink appreciably for a period of 12 to 18 days. This factor, of course, is not effective where a thin deposition of fresh solids is made over a large area of ripe sludge. In the latter case the fresh solids almost at once become engulfed in the mass below so that it is, in fact, difficult to secure any sample of the freshly deposited solids from a digestion chamber which is working properly. (The latter statement was also expressed by Dr. Imhoff.) A longitudinal section of the tank is shown in Fig. 7, indicating the sloping inlet and the distribution of the solids under three conditions: (*a*) The plant as originally constructed, with six tanks operating; (*b*) only three tanks operating so as to increase the velocity by cutting down the total cross-section; and (*c*) a distribution which it is hoped to attain by raising the slots (cutting down the cross-section of the tank at the point of inlet), using submerged orifice inlets, and making possible the use of six tanks in parallel.

Similarly, the distribution across the section of the tank is subject to some measure of control. As Mr. Gavett points out, plain mechanics would

account for a better distribution of solids with the increase of the distance through which they have to fall after passing the slot.

In Fig. 8 is shown the cross-section of the Plainfield tanks as built and as altered to meet the criticisms of Dr. Imhoff. The inlet will be through a submerged orifice placed at the bottom of the old slope, so as to come within the restricted cross-section in the hope that the increased velocity will carry some of the solids forward to the second and third hoppers. The bottom of the bulkhead containing the orifice will be movable for the removal of solids lodged behind it. The Operating Committee of the Plainfield Plant was impressed with the possibilities of relief by using these ideas and felt that in the interests of all concerned the proposition deserved a fair trial, so that it has authorized the alteration of one tank on an experimental basis.

With these changes made, Dr. Imhoff expects no further difficulty in operating the plant satisfactorily; he specifically includes in his definition of satisfactory operation the elimination of the excess labor, and agrees with the speaker that an Imhoff tank which operates by virtue of excessive labor or of mechanical devices for keeping down scum is not a satisfactory installation.

Dr. Imhoff did not advance any theory in support of the changes advocated, and his recommendations were not accepted without considerable study. The available data for this study included:

1.—Operating data on the Plainfield tanks over a period of nearly seven years, showing: (*a*) The distribution, which had been very poor at first, to be somewhat improved by operating only half the tanks at any one time, and the improved distribution of solids to result in a greatly improved digestion; (*b*) the volumes of solids deposited, calculated from cone glass readings over these years; (*c*) a check of these volumes by means of the vertical deposition of solids in buckets hung in the settling compartments at various points*; (*d*) the quantity of ripe sludge removed in 7 years; and (*e*) the length of time required to digest solids to a 90%-95% sludge, the best sludge obtainable in these tanks (the time equals about 30 days).

2.—Published data from various sources indicating that digestion tanks in good working order may be expected to evolve gas in about the following proportions: CO_2 — 10% or less, CH_4 — 85%, N, by difference — 5% or less.

3.—Data furnished by the New Jersey Sewage Experiment Station as to the proportion of different gases evolved at different stages of the process of digestion. Although these data were somewhat erratic and not very satisfactory in the opinion of the chemist on account of the difficulty encountered in collecting the gas, due to stoppages in the small tubing, they were considered to be sufficiently accurate for the purpose in hand, to show: (*a*) that a considerable lapse of time is required to get the proper CH_4 digestion under way; (*b*) that such lapse of time is roughly commensurate with the time required for minimum satisfactory digestion of sludge (30 days); and (*c*) that the final extremes are reached only after a considerable further lapse of time (about 90 or 100 days).

In running one tank continuously to ascertain the ultimate outcome of such treatment, the Chemist from the Experiment Station, Mr. Leslie

* These data were furnished by the New Jersey Sewage Experiment Station.

Campbell, found that the disturbance in that portion of the digestion chamber above the sludge proper increased until the quantity of solids in all the liquid within the compartment reached more than 3 per cent. This condition continued until about the twenty-fifth day when there was a distinct change in conditions, temporarily. It seemed as if the process was just reaching some culminating point, but evidently the compartment was not large enough to permit the completion of this change and after a brief interval the whole mass was again thrown into a state of suspension.

Weighing all the evidence in hand it seems clear that (1) the slot of an Imhoff tank should be placed with due consideration of the progress of digestion which will have been attained at the time when the freshly deposited solids (without shrinkage), have accumulated to a depth equal to the distance between the bottom of the tank and the slot; and (2) that the horizontal dimensions of the tank should be such as to distribute the solids as evenly as possible.

In Figs. 9 and 11 are indicated graphically some of the results of these studies. The curve showing the progress of digestion of a unit volume of fresh solids (Fig. 11), starts daily on that day's deposition of solids, to end 30 days later in the shrinkage of these solids to 40% of the original volume. If the compartment is sufficiently large, so that the digestion can proceed further, it seems fair to assume that, in the course of a period of time which is not at present definitely known, the moisture content of the sludge will be changed from 90%-95% to 88%, 85%, or even 80%, resulting in a further shrinkage of volume. The broken arrow (Fig. 11) indicates the general direction of such shrinkage.

In Fig. 10 appears the summation of the volumes of daily accumulations of solids, each decreasing according to age, as compared with the volume of uncontracted solids deposited. This gives an interesting picture of the effect of even a small change in the position of the slot. It is the speaker's belief

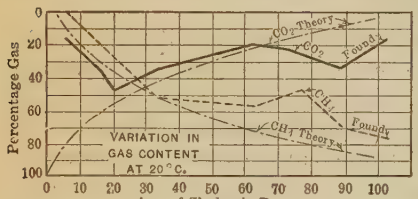


Fig. 9.

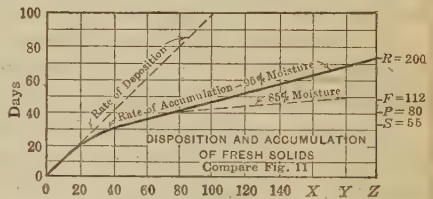


Fig. 10.

that the slot at Plainfield is just within the very critical zone where the accumulation curve begins to assume its final direction, and that the Fitchburg slot is placed just far enough above this critical zone to make the difference between success on the one hand and failure on the other.

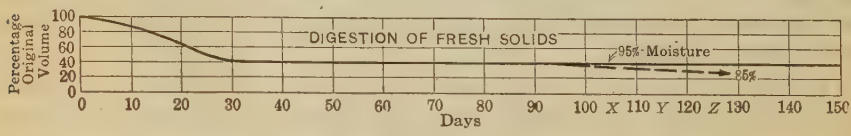


Fig. 11.

These two plants have several points in common which make them specially adaptable for comparison, as shown in Table 18.

TABLE 18.—Comparison of Plants.

	Fitchburg.	Plainfield.
Flow, in million gallons per day	3.4	3.4
Population	38 000	40 000
Sludge space, in cubic feet per capita	1.47	1.22
Domestic sewage solids, in grammes per day per capita.	54	53*

* Allow 20 grammes for storm water at Fitchburg.

It would seem fair, therefore, to assume for purposes of comparison that they receive approximately equal volumes of domestic sewage solids per capita. At Plainfield, this amounts to 0.0305 cu. ft. per capita, or 30.5 cu. ft. per 1 000 population. The greater weight of suspended solids retained at Fitchburg would be largely made up of heavy minerals. The space occupied by such minerals is 1% of the freshly deposited solids, calculated from comparison of solids by weight and by volume.*

Assuming an equal distribution of solids in both cases, this volume would reach the bottom of the neutral zone (without any contraction) in 48 days at Fitchburg and 40 days at Plainfield. These periods are for the deposition of fresh, uncontracted solids. From Fig. 10 the actual capacity for sludge of the consistency found at Plainfield is 112 days at Fitchburg and 80 days at Plainfield.

What about the difference in density of the two sludges? It is the speaker's belief, although at present he has no positive proof of it, that the difference is due to a time factor as indicated on Figs. 10 and 11. In a recent article† he pointed out that this time period was certainly greater than 60 days and probably greater than 90 days (corresponding to the final adjustment of the proportions of gases evolved). If this critical point falls between 80 days and 112 days, the capacity of the Fitchburg sludge compartment becomes something like 160 or 170 days, while that at Plainfield remains at 80 days.

The Schenectady and Rochester plants are not so susceptible to comparison; and yet the weight of suspended domestic solids is near enough to justify the application of the same unit volume of solids in all four cases to see where these two plants fit into the picture. Schenectady has 33 days' capacity for uncontracted solids and Rochester 72 days. The position of the bottom of the neutral zone in terms of effective capacity has been indicated by the initial letter of the city at the right in Fig. 10, as follows: Schenectady, 55 days; Plainfield, 80 days; Fitchburg, 112 days; and Rochester 200 days.

All this calculation has been conditioned on an even distribution of sludge from end to end of each tank. As has been already shown this is far from the fact at Plainfield and, presumably, at Schenectady, where the communication

* Plainfield Laboratory data.

† *Public Works*, Vol. 54 (1923), p. 363.

between hoppers is much restricted. This suggests another factor in favor of the two successful tanks.

The author, calculating from the weight of suspended solids, has taken 1.83 cu. ft. per person (60 days at 0.0305 cu. ft. per capita) as the required sludge space. In calculating from the volume of fresh solids deposited, the speaker feels that 60 days' capacity of such solids should be the minimum allowance. It is true that Fitchburg is successful with an allowance of only 48 days, but this is too fine a line to draw in designing a plant. Again, Mr. Eddy computes 2.75 cu. ft. of ripe sludge per person to be removed per annum. The result of actual measurement at Plainfield is 0.1 cu. yd. per person per year, or 2.7 cu. ft. It is strange, however, that although these figures check so exactly, Mr. Eddy has calculated on a sludge containing 15% of solids and the speaker's experience has been with sludge containing barely half as much.

It is hard to explain this discrepancy. To designate the condition of a sludge in percentage of solids is inadequate. There is no escaping the mathematical correctness of this method of expression, but the idea that a sludge of 15% of solids simply contains twice as much solids per volume as a sludge of 7.5% of solids is an incomplete grasp of the situation. There is something more than a mechanical concentration. Sludges of less than 6% solid content may be considered as merely a mechanical mixture, but when through the natural process of digestion the solid content reaches 7%, there is a definite change in the character of the sludge; it might almost be stated "in the structure of the sludge", but up to this point the sludge has no structure—it is a heterogeneous mass of solids and water. Above 7% it may be said to assume a sort of structure—there is a re-arrangement of the particles, the mass becomes viscous and this results in a capacity to hold great volumes of gas. This change is progressive through a range from 7 to 12% of solids at least. Although nothing better than the percentage statement of solids is offered at this time, such an expression is inadequate and it is to be hoped that something better may be developed.

It is reasonable to suppose that the considerable volume of gas which the heavier sludges are capable of entraining could materially effect the volume of space occupied by a given weight of solids in a given dilution of water. This matter might well be added to the list of subjects on which more light is required.

David A. Hartwell,* Esq. (by letter).—The author has covered his subject so thoroughly that there seems little to be added; there are a few matters, however, that may with interest be further discussed. The writer has a feeling that the same type of tank construction might act differently in different localities due to varying conditions, such as the climate, the composition of the sewage, and the theory of the operator as to the causes of certain objectionable conditions and the methods of correcting them.

In designing the Fitchburg tanks provision was made for treating the sewage from a population of 55 000 with a theoretical retention period of 3 hours. During 1923 the retention period varied from 10 hours in December

* Commr. of Public Works and City Engr., Fitchburg, Mass.

to less than 5 hours in March, the weighted average for the year being 6¼ hours, as compared with 6½ hours in 1922 and 8½ hours in 1921. Possibly the long period of retention in the sedimentation compartment at Fitchburg may cause a settlement of certain solids that would not be effected during a shorter period and this may have considerable effect on the process of digestion.

In summer the water is cooler in a deep well than in a shallow one. Is it not reasonable to assume that the temperature of the sewage in a shallow tank would be affected more by changes in the temperature of the air than the sewage in a deep tank, being colder during the winter months and warmer during summer months? If this were a fact as well as a theory, would not digestion in a deep tank likewise be more active during the colder months and less active during the warmer months? According to Fig. 1, an increase of 3° in temperature in the digestion compartment would entail a considerable increase in the activity of the digestion. Recent tests at Fitchburg made after a period of zero weather for several days indicate a temperature between 44° and 45° in the digestion compartments of all the Imhoff tanks at a point about 4 ft. below the slots. This is probably a minimum temperature under normal winter conditions.

It has been the custom at Fitchburg to draw sludge from the tanks as early as conditions on the drying beds warrant. This removes any digested sludge early in the season and leaves the digestion compartment in the best possible condition for the maximum work to be done during the warm months. In 1921 as much sludge was removed in April as during any other month except June and October; in 1922 the April sludge removal was only exceeded by that of July; and in 1923 that in May was greater than in any month except June and July. The removal of sludge from the tanks as early in the season as possible has been a large factor in the elimination of foaming during the last three years.

In the first years of operation of the Fitchburg plant the disposal of the secondary tank sludge was a serious problem. The design of the plant made provision for pumping this sludge into the influent pipe to the Imhoff tanks, in the expectation that it would mix with the sewage, to settle and be digested with the sewage sludge. This method delivered the secondary tank sludge into the sedimentation compartments of all the tanks while they were in normal operation, which procedure was objectionable on account of (1) the light flocculent foam that covered all the tanks for a number of days, (2) the reduced efficiency of the trickling filter and (3) the unsatisfactory final effluent. A later method was to pump the secondary tank sludge into one or two tanks instead of all five. This necessitated shutting down the whole plant and turning all the sewage into the river while the secondary tank sludge was being pumped—an objectionable method. Finally piping was installed to carry this sludge from an outlet 18 in. below the slots directly into the digestion compartment of one tank while the whole plant is in operation. The sludge can be delivered to any or all of the hoppers of this tank. This method has now been in operation for nearly three years with highly successful results. There has been absolutely no foaming in this tank, which has digested all the sludge from the secondary tanks. The mixture of secondary tank sludge with sewage sludge is more

rapidly digested than sewage sludge alone and there is a great increase in the volume of sludge pumped yearly from this tank. During the last three years 43% of all the sludge pumped from the five Imhoff tanks has been pumped from this one tank.

MORRIS M. COHN,* ESQ. (by letter).—The operation of the Imhoff tanks at Schenectady always has been and, no doubt, always will be the cause of much trouble. Only by the application of much labor and time can they be made to function properly and as yet no means have been discovered that will produce normal operation.

It appears from experience, that, if left without attention, the tanks will not foam. The scum blanket will increase in depth and dry out, producing an apparently foam-proof layer through which the gases cannot escape. Furthermore, any attempt to draw sludge will meet with failure. Hosing of the scum accumulation will drive down the mass and make possible the drawing of a mixture of sludge, scum, and water. This is proof that the sludge obtained cannot be truly normal. A well-behaved Imhoff tank will always have the oldest sludge immediately tributary to the sludge pipe; but the constant agitation of the Schenectady sludge gives a mixture of old sludge, moderately fresh sludge, and undigested particles just deposited. Such a mixture can never attain the condition that typical sludge should have when drawn for dewatering. It is only at the height of the warm season, when digestion is rapid, that this mixture reaches a reasonably good condition. This heterogeneity accounts for the fact that analyses show only a slight variation in the composition of the sludge from the first drawing of the season until the final drawing in the fall.

Thus, it is the formation of scum rather than of sludge that can be held accountable for the tank difficulties at Schenectady. Two conditions may cause the rise of the solids deposited in the sludge compartment of the tanks: (1) the solids may be lighter than the sewage, due either to their own character or to the effect of some outside agency; and (2) the entrained gases may be held so tenaciously that they do not escape when the solids rise in the gas vents, thereby giving the mass a specific gravity less than that of the liquid. It is impossible to ascertain which of these factors is the cause of the difficulty, or whether both causes contribute to the general condition of the tanks; it suffices that the net result is the same whatever the specific cause.

During the summer of 1922 the majority of the sludge samples drawn for test had specific gravities less than water. Furthermore, a thorough agitation of the samples failed to liberate the gases sufficiently to increase materially the density. Mr. Eddy suggested that the sludge be placed in a freezing solution and then agitated to free the gases. Only after such treatment was it possible to obtain gravities greater than 1.0. This condition indicates that the sludge in the hoppers is composed of particles of low specific gravity easily buoyed up by relatively small quantities of entrained gases.

There has always been a tendency for the fresh solid particles deposited in the hoppers to rise to the surface of the gas vents. On the resumption of

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operations in 1921, following a period of idleness for repairs, the tanks started in a clean condition. Despite the fact that operation was commenced the first week in November, when the temperature was quite low, the first few days of operation produced a covering of fresh solids in the vents of the tanks. This rise of the solids may be attributed in part to the formation of gas, but also in part to the inherent lightness of the solid particles themselves, because of their own characteristics or because of some outside agencies.

It is not easy to ascertain the cause of the solids and sludge rising in the vents. One thing is certain, namely, that at times the city sewage contains a great deal of grease and oil. During one of the worst days there has been removed as many as sixty wheel-barrows of black oil and grease from the inlet to the tanks. As further proof that the sewage flow contains a great quantity of oil and grease it can be stated that the exposed concrete about the tanks is stained by grease and that a thick gelatinous grease layer is found in the effluent channels and dosing tanks. Grease balls are a great cause of nozzle clogging on the trickling filters, and the spindles of the Taylor nozzles are always coated with a heavy grease layer. The bottles in which the daily sewage samples are collected and in which the composite weekly samples are made up, are oily in appearance. Further, the sludge after de-watering has an oily smell. Although most sewages contain a certain quantity of oils and grease, still it may be true that the raw material at Schenectady contains the same elements in an excessive degree. This grease may produce a tenacious, waterproof coating to each particle of sludge and make it impossible for the entrained gases to break free, allowing the mass to settle. These small particles may be likened to gas-filled balloons that are set free and fly to a great altitude. As the balloons rise, the gas tends to expand, due to the decrease in atmospheric pressure. If the material of which the balloons are made is weak, this expansive force will burst them and they will fall to earth, but when the enclosing shells are so tough that the internal pressure will not rupture them, the balloons will stay suspended as do the particles of Schenectady sludge. The action of the sludge samples of 1922, referred to previously, proves either that the gases were held so tenaciously that beating would not release them, or that the gas formation was so rapid as to offset the rate of release. On the other hand, it may be that the gases are under so low a pressure, due to the shallowness of the Schenectady tanks, that they would not have sufficient expansive force to break free from a sludge that offered even less resistance than the Schenectady sludge. At any rate the grease theory appears to warrant consideration.

In reference to the quantity of soap curd produced in this city, it will be interesting to quote from an investigation of the hardness of Schenectady tap water carried out by the writer in 1921 for the purpose of determining the feasibility of installing a softening plant. By a series of laboratory experiments the waste ratios for representative soaps were determined. This waste ratio is the ratio of the quantity of soap required to soften the water to the total quantity used. It is sufficient to state that the average ratio for all soaps was found to be about 6% with a water of the hardness of Schenectady's

supply. It can readily be seen that the soap curd produced by 6% of all the soap used by the population tributary to the plant will be very large in the aggregate. On the basis of the hardness of the water in parts per million, this quantity per 1 000 000 gal. of sewage at Schenectady would be twice as great as at Rochester and thirteen times as great as at Fitchburg. This curd, no doubt, acts as a coating for the sludge particles and produces the same effect as oils and greases. As Mr. Eddy states, it surely adds to the load of suspended solids on the units. It certainly appears that the great quantities of soap curd must be the cause of some of the difficulties at Schenectady.

It is of interest to ascertain whether or not there is any inhibitory action produced on the sludge by the materials already in the sewage, or introduced into it during the ordinary course of treatment. The sewage contains a great deal of mash and grain which comes, no doubt, from the illicit manufacture of liquor. Many sewers in the city have been found to be completely choked with this material. It seems that all this matter finds its way into the gas vents and cannot be driven down for any length of time. One-half hour after the scum blanket in a gas vent has been driven down by the use of a 1¼-in. stream of water, there will appear a layer of at least ½ in. of swollen and puffed grain on the surface. Below this, the scum builds up, but never yet has the grain failed to rise before the sludge. A distinct sour odor was noticed during the spring of 1924 when the vents were first hosed. There is no question but that true fermentation is taking place in the sludge hoppers, and it may be that the carbon dioxide emitted retards the digestion of the sludge.

The effect of the water introduced into the vents during the process of hosing is debatable. It might be expected that the dissolved oxygen would tend to stimulate the kind of biological action which produces carbon dioxide and that this would affect adversely the digestion process. An experiment in which sewage was used to drive down the scum on one tank and water to perform the same duties on the other tanks under the same conditions disclosed the fact that all the tanks acted similarly in regard to the formation of scum. It can be said, nevertheless, that the addition of water to the scum blanket does seem to produce the phenomenon of foaming. At Schenectady, this seems to be due to the softening of the scum layer rather than to any gas action. It appears that the dried scum retards the passage of gases and exerts a downward pressure that offsets the tendency of the vents to foam. Under this condition a great deal of sludge finds its way into the sedimentation chambers through the slots, the material following the path of least resistance.

A few other phenomena have been noticed by the writer, that seem to bear out the fact that the Schenectady sludge has an inherent tendency to rise. When the tanks were cleaned during the spring and early summer of 1923, a small quantity of sludge was left in the bottom of the hoppers for seeding purposes. It was noticed that in a few hours the water had settled out of this residue leaving the sludge as a heavy scum on the surface. This was so marked that the material was stirred well before samples were taken

for analysis. There is another point that may be of interest: When during the summer of 1923 sludge was drawn from the tanks immediately after hosing, in many instances there would be a good flow of moderately heavy sludge for a period, followed by a black septic liquid. No more sludge could be drawn despite the fact that the gas vents were clear. After a few hours, scum would again form and by the end of 24 hours a heavy deposit would have gathered in the vents. The question arises as to where the solids were at the time of drawing. It might be expected that the driving down of about 3 or 4 ft. of scum would produce a sludge deposit tributary to the sludge pipe that could be drawn at will. This was not the case, and it certainly looks as if the buoyancy of the scum was not destroyed by the action of the 1¼-in. stream of water under 60 lb. pressure. Most of the scum must have been suspended somewhere between the surface of the vents and the bottom of the tanks.

An investigation was begun early in 1924 to determine the reaction or hydrogen-ion concentration of the sewage liquid drained from the material in the sludge and scum compartments, and of tank effluent. Table 19 gives the results of the pH determinations to date.

TABLE 19.—Hydrogen-Ion Concentration at Schenectady, N. Y., 1924.

Date.	Raw sewage.	Tank effluent.	Drain liquor from sludge.
April 11, 1924	7.1	6.9	....
April 17, 1924	6.8	6.6	7.3
April 26, 1924	6.8	6.6	7.3
May 2, 1924	....	....	....
May 9, 1924	7.2	6.9	....
May 16, 1924	7.3	6.8	....
May 21, 1924	....	....	7.9
May 30, 1924	8.0	6.8	....
June 3, 1924	....	....	7.8

There appears to be a distinct increase in the acidity of the sewage in its course through the tanks, and the sludge liquor appears less acid than either of the other samples. If the pH values are indicative of the bio-chemical changes occurring in the tanks, it will be of interest to watch the effect of the later warm weather on them.

TABLE 20.—Temperature of Schenectady Sewage, in Degrees Fahrenheit.

Month.	Air.	Crude sewage.	Tank effluent.	Sludge compartment.
1923				
December	36.6	52.5	51.9	50.0
1924				
January	27.6	51.1	50.3	50.0
February	22.2	49.1	48.3	48.0
March	36.8	48.9	47.9	48.0
April	46.8	48.9	48.5	48.5
May	55.8	52.0	51.5	53.0

The question of temperature effects on the operation of the Schenectady tanks should be answered by the record of the temperature of the sludge in the

digestion compartments now being obtained by the use of a compensated recording thermometer. The average monthly temperatures of air, crude sewage, tank effluent, and sludge compartment contents since the installation of the gauge is shown in Table 20.

The first signs of any distinct gas formation in the vents were detected in May, during which month the highest temperature recorded for the material in the sludge compartments was 57° Fahr. It is certain that little digestion can go on at temperatures lower than those experienced in May. In June, with an air temperature of 80° Fahr., the temperature in the sludge compartment reached 61° Fahr. Evidently, the period of active digestion of sludge will be short, and the great mass of the deposits will produce much gas. This large-scale formation of gas may be the cause of the foaming tendency that is already becoming evident. The continuation of the temperature record should throw some light on the effect of temperature on the digestion of sludge in the Imhoff tanks at Schenectady.

A. P. Learned,* M. Am. Soc. C. E. (by letter).—In this instructive paper on Imhoff tanks, the author mentions the mechanical agitators at Enid, Okla., and at Newton, Kans., used to break up the scum in the vent chambers, and to overcome the nuisance caused by foaming. The writer visited both these plants on August 5 and 6, 1924, and hopes that the following description of these agitators and of the results obtained may prove of interest and benefit to members interested.

The Imhoff tank at Enid was designed for the sanitary sewage from a population of 20 000; the present tributary population is 16 000. Two years ago (1922), one year after the plant was completed, the Imhoff tank foamed and the City Department in charge of its operation devised a mechanical agitating device that has since satisfactorily broken up the scum so there has been no further foaming and the scum has been practically overcome. Table 21 includes general data regarding this plant and other plants described in this discussion.

The tank at Enid is 54 ft. long inside and has three vent chambers running its entire length. Across the tank at the top are four tie-beams dividing each vent chamber into five compartments, that is, from the standpoint of installing agitators. Thus, there are fifteen agitators each of which can be removed without interference with any other.

As shown in Fig. 12, each agitator consists of a 1-in. galvanized-iron pipe shaft placed horizontally and in length approximately 6 in. shorter than the opening between the beams. At two points in its length are inserted ordinary crosses holding, on one side, a nipple approximately 4 in. long and, on the other side, one approximately 2½ in. long. Pieces of pipe approximately one-fourth as long as the opening between beams, attached to the nipples by tees, complete the framework. The two crosses on the main shaft are set with openings 90° apart to make the paddles balance. The agitator thus has an influence area 12 in. in diameter. The main shaft of the agitator is inserted on each end into tees of larger size for bearings. Ordinary floor-plates are

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keyed on the main shaft to keep it from moving horizontally. The main shaft is just at the top of the normal water level of the tank and the water lubricates the bearing. The whole mechanism is attached by light bolts to the concrete on the top of each side of the vent chamber. The apparatus is operated by a main through a chain drive and an intermediate main shaft running the length of the vent chamber and supported on shaft hangers, giving a speed of 4½ rev. per min.

TABLE 21.—Data Respecting Imhoff Tanks.

	Emporia, Kans.	Enid, Okla.	Newton, Kans.	Whittier, Calif.
Population designed for	13 250	20 000	8 000	10 000
Tributary population	12 000	16 000	8 000	10 000
Separate sewers	Yes	Yes	Yes	Yes
Freshness of sewage	Fresh	Fresh	Fresh	Stale
Free-board	1.5 ft.	2.0 ft.	1.0 ft.	2.0 ft.
Depth top of water to bottom cone	26 ft. 7 in.	25 ft. 4 in.	25 ft. 0 in.	29 ft. 7 in.
Total length inside	44 ft. weir built in.	54 ft. weir, built in.	28 ft. 4 in.	52 ft. 0 in.
Number of settling compartments	2	2	2	2
Number of vent chambers	1	3	1	3
Width of vent chambers	2 ft.	Two 1 ft. 4 in. One 1 ft. 8 in.	2 ft.	Two 2 ft. 10 in. One 4 ft. 8 in.
Percentage (area of vent chamber ÷ area of settling compartment)	10	27	13	65
Width of sludge chamber	16 ft.	22 ft. 4 in.	14 ft. 0 in.	26 ft. 0 in.
Depth water to slot	10 ft. 6 in.	11 ft. 4 in. upper 12 ft. 9½ in. lower.	10 ft. 3½ in.	9 ft. 7½ in.
Number of sludge hoppers	2	3	2	2
Width of settling compartments	10 ft.	8 ft.	7 ft. 8 in.	7 ft. 6 in.
Depth of hoppers at bottom tank	4 ft.	6 ft. 0 in.	2 ft. 9 in.	6 ft. 0 in.
Depth of water below gas deflection beam, to top hopper in bottom of tank	10 ft. 7 in.	6 ft. 7 in.	10 ft. 6 in.	12 ft. ½ in.
Sludge storage capacity per capita	0.7 cu. ft.	1 cu. ft.	0.57 cu. ft.	2 cu. ft.
Reversible flow	No	Yes	No	No
Screening	Bar screen, ¾ in. clearance	Bar screen, ¾ in clearance	Bar screen, 1 in. clearance.	None
*Retention period (average flow)	1 hour.	1 hour.	1 hour.	66 min.

* Average flow assumes all sewage flowing off in 16 hours.

The operator uses the agitator ½ hour in the morning when he first comes to the plant, 15 min. at noon, and 15 min. just before he leaves at night. At each treatment, the small amount of matter not easily settled or digested, such as seeds, vegetable leaves, string, match sticks, etc., is skimmed off.

There has been no foaming, and no great amount of scum since the installation of the agitator. The effluent is dosed on a sprinkling filter and it is maintained that since the installation of the agitator the odor about the plant has been materially decreased, especially during the sprinkling on the filter. Practically all agree that there is usually much more odor noticeable during periods of foaming. It is probable that the free ebullition and escape of gas

reduces the concentration and perhaps the formation of stronger gases that occur during foaming periods.

The operator advises that the power required to keep the agitators going is less than 1 h.p., but that it probably requires between 2 h.p. and 3 h.p. to start it. The cost of operation for power averages from 10 to 13 cents per day. The materials cost $350 exclusive of the motor and the speed reduction set, and the cost of installation was less than the cost of the materials.

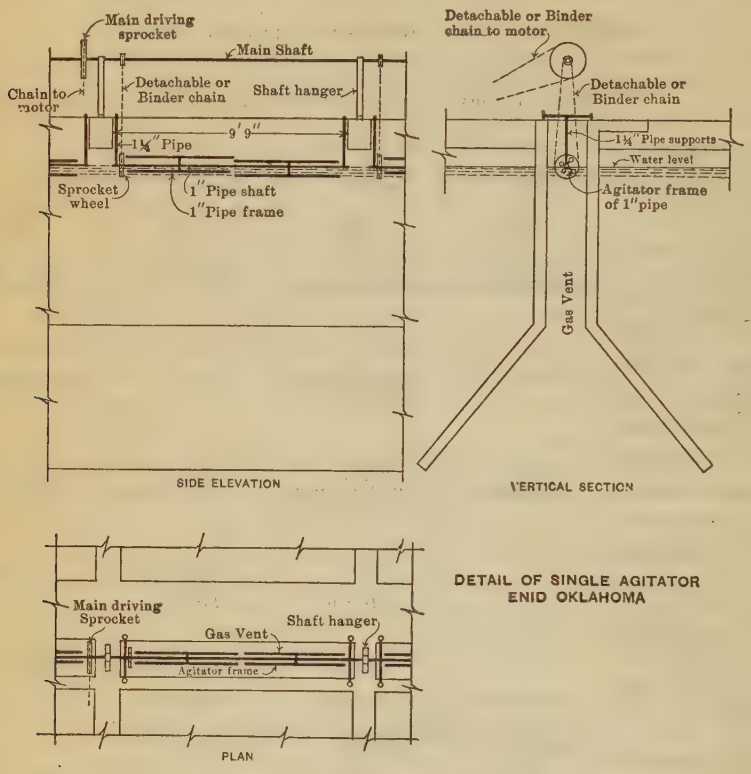

FIG. 12.

The plant produces a good effluent and stable sludge. Sludge is removed in small quantities often during the summer or drying season. The sewage which is relatively fresh as it comes to the plant passes through a fixed bar screen having a ¾-in. space between bars and is pumped by centrifugal pumps passing 1-in. material into the tank. Provisions are made to reverse the flow in the tank.

The pump suctions are always submerged. Some oils and fats collect in the pump pit. The operator has built a skim weir well above the ordinary operating height in the sewage sump and, at times, raises the float-actuating devices above this height allowing the pit to fill up to the skimming weir. Then he skims off the grease and fats, thereby relieving the tank of all such materials, except those that will not readily separate.

The Imhoff tank is practically free from scum in the vent chambers and has no appreciable floating particles in the settling compartments. In this tank, the lower slot of the settling compartments is constructed by the overlapping of the sloping bottoms and is apparently satisfactory, although it is the writer's opinion that the double slot with the triangular beam at the bottom is better.

At Newton, Kans., the scum agitator (Fig. 13) is of different design and operation, but is satisfactory. In the tank there is only one scum compartment, 28 ft. long and 2 ft. wide, and the agitator consists of four mill screws approximately equally spaced in the chamber, mounted on pipe-frames, two on each frame, and each frame mounted at one point on a walking-beam. Each frame is balanced so that the agitators travel equal distances between 12 and 18 in. vertically. The mill screws extend about 4 ft. into the water. The walking-beam is connected, as shown in Fig. 13, by speed-reducing equipment to the shaft of a vertical motor driving one of the sewage pumps. The speed of the agitator is between 4 and 4½ rev. per min. It operates whenever the relief pump to which it is connected is used, or considerably more than 1 hour per day.

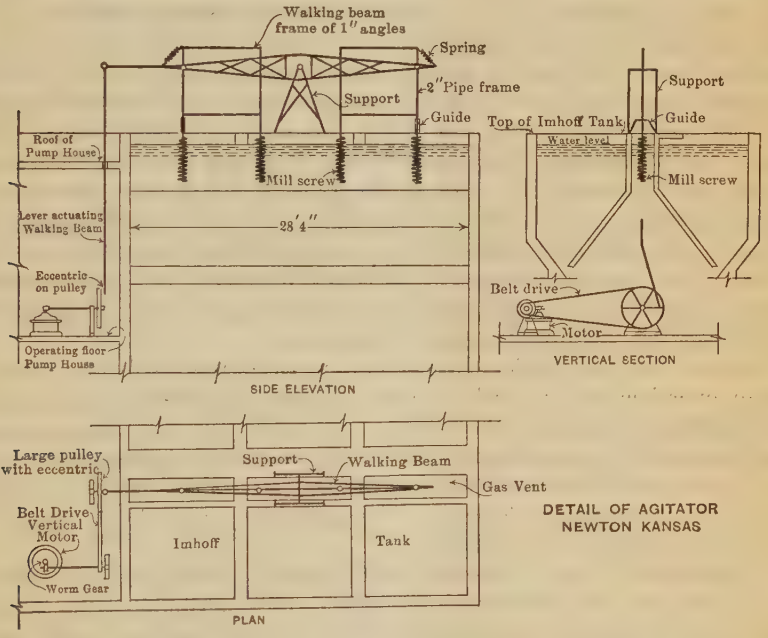

FIG. 13.

The writer noted some scum in the vent chamber, between the agitators, but it was not more than 2 in. thick. When the installation was first made the plant was foaming badly and the sludge compartment was well filled. The official in charge, however, felt that the agitator would free the gas and the foaming would cease and no sludge was withdrawn for more than a month after the agitator was started. No foaming occurred during this month, or

subsequently. The sewage disposal plant at Newton is of the same type of units as at Enid and satisfactory results are obtained. Sludge is not removed from this tank at less than intervals of three months.

In his discussion, Mr. Downes states that Dr. Imhoff agrees with him, "that an Imhoff tank which operates by virtue of excessive labor or of mechanical devices for keeping down scum is not a satisfactory installation." This may be true, but in both the plants herein described, satisfactory results are obtained and the mechanical breaking up of the scum is inexpensive and effective.

The characteristics of the Imhoff tank built at Emporia, Kans., are shown in Table 21. This tank has no mechanical agitator at present. It was foaming on August 6, 1924, at the time of the writer's visit. The Superintendent of the plant expects to install a mechanical agitator to remove this difficulty, obtaining power for its operation by means of a windmill, thereby reducing the operating expense to a minimum.

It will be noted in Table 21 that the capacity of the digestion chambers in each of these plants is materially less than that recommended in the paper and in the discussions. However, foaming difficulties have been experienced in the tank at Whittier, Calif., in which the sludge capacity was 2 cu. ft. per capita and the settling compartment is designed for 40-min. retention of the maximum flow. The foaming occurred shortly after the plant was completed, considerably previous to the time that the digestion chamber would have been practically filled with sludge and before the condition had become critical. In the plant at Whittier, the free-board was 2 ft,, and the foaming was quite violent and continued. It could hardly be stated that here there was not adequate sludge capacity, and it seemed that possibly the fact that the sewage was quite old and fairly septic before it reached the tank might have contributed to the excessive amount of scum which formed in the vent chambers. In this instance, there was also a large percentage of scum area, amounting to 40.8% of the entire top surface of the tank.

Mention has been made of the fact that most frequently the scum consists of unbroken sewage, such as paper, strings, vegetable remains, rags, etc. According to the writer's observation, the only scum that has become distinctly hard and impairs the free escape of gases of decomposition, is that in which decomposed fecal matter forms the matrix and seals the pores between the slower decomposing elements which normally break down through fermentation rather than putrefaction.

At Goodland, Kans., in a small tank excessive foaming occurred within a month after the plant was put in operation and immediately after the sewer system was completed, when the amount of sewage flow was small. As the tank foamed, the material appeared to have no particular body, but after it flowed over the settling compartments and dried, it was quite tenacious and had the appearance of thoroughly digested fecal material. After the tank had a heavier sewage load, this ceased and has not occurred to any marked degree since.

During the spring of 1924, at Broken Bow, Okla., a plant which had been in operation somewhat more than a year, and which up to that time had

given no difficulties suddenly experienced the rising of most of the sludge in the lower compartment which foamed over the vent chamber, thoroughly covering the settling compartments at a rate faster than could be handled by the operator who occasionally visited the small plant. This occurred at approximately the same season in the spring as the semi-annual turn-over in water reservoirs.

It has seemed to the writer that there are so many different conditions under which foaming occurs that no single reason can be assigned for it. It is felt that the septic condition of the sewage at Whittier might have been conducive to the excessive forming of scum and the entraining of gas to the extent of creating a foaming condition. However, this septic explanation would not apply to the other plants described unless possibly to the small plant at Goodland. In all the plants herein discussed, the sewage has been strictly sanitary, with no street washings or trade wastes that are particularly difficult to handle in the normal sewage disposal works. The mechanical devices mentioned have effectively removed the trouble of foaming and possibly have resulted in satisfactory operation with the plants the sludge compartments of which were not as large as has been generally accepted as proper in these discussions. It has seemed also that even with a large sludge compartment, freedom from foaming is not assured, as evidenced by the plant at Whittier, where, due to climatic conditions and the rapid and continued decomposition of sewage, there would not be the necessity for long storage of deposits in order to insure its proper digestion.

In the section of the United States considered there has been a tendency (probably more serious in consequences than in the Eastern part) to treat a sewage disposal plant as a utility which functions with practically no attention. It is the writer's opinion that greater pressure must be brought to bear on all communities in order to make them realize that a sewer system or a sewage disposal works is as much a public utility as a water-works or lighting plant; that although it may not require constant attention, yet it requires regular and competent supervision for satisfactory results.

If the design is reasonably ample as to the capacity of the digestion chamber and vent area, the writer believes that foaming probably is dependent on the character of a sewage at the particular time and is distinctly an operation problem.

KARL IMHOFF,* M. AM. SOC. C. E. (by letter).—This paper is extremely interesting; it is certainly the best and most complete treatise ever written about two-story settling tanks. Although the writer fully agrees with the deductions drawn he would like to expand them in some particulars but only according to experiences gained in Germany.

Character and Composition of Sewage.—It is very seldom that the sewage to be treated is already existing or its quantity and composition exactly known before the plant is designed. During the early operation, the sewage usually undergoes great changes and, therefore, the engineer is obliged to design his plant according to given data, that is, the number of inhabitants and the

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quantity of water. Besides, he must endeavor to estimate to the utmost all other conditions influencing the quality of the sewage. Influences which increase the sludge quantity and which—for other reasons—require a large capacity of the digestion tank, are:

1.—Storm water, in combined system (50%);
2.—Absence of a grit-chamber;
3.—Absence of a fine screen;
4.—Low average temperature—long winter;
5.—Industrial wastes; and,
6.—Extravagant habits of the people (home breweries, large consumption of fat, paper, etc.).

Temperature of Digestion.—The methane fermentation depends on: (1) The seeding and mixing; and (2) the temperature.

The author is perfectly right in devoting the greatest attention to the temperature. The artificial seeding and mixing is of the utmost importance in plants having separate sludge digestion chambers, as it shortens materially the time of ripening (Birmingham).

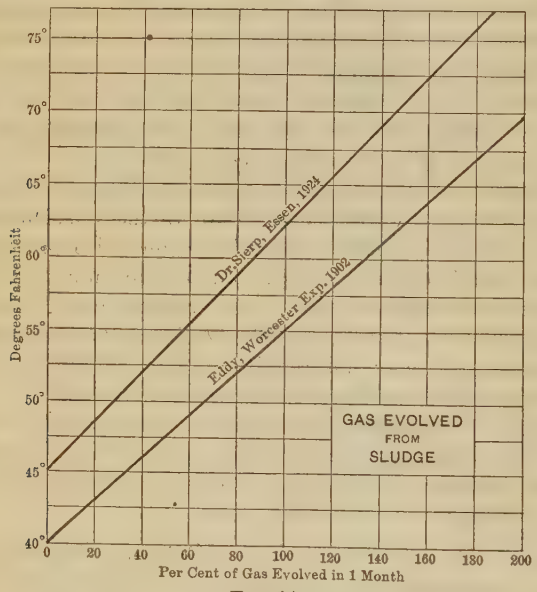

FIG. 14.

In two-story tanks the seeding and mixing is done automatically; but the best mixing is of no effect in case of the slightest change in temperature. Mr. Eddy submits a gas curve (Fig. 1), based on data published in 1902. This curve shows that the production of gas up to a temperature of 75° (Fahr.) is proportionate to the temperature. In Fig. 14 is shown another curve, based on Dr. Sierp's (Essen) latest experiments with separated sludge tanks. This latter curve shows some slight differences in the values as compared with that of Mr. Eddy, but also indicates that gas production varies as the temperature.

Therefore, as a general rule, the proposition of Mr. Eddy must henceforth be considered valid, namely, in designing sludge digestion tanks the temperature must be estimated as exactly as possible. This is relatively simple, because the temperature in the digestion chambers of two-story tanks generally accommodates itself to the temperature of the sewage, and in separate sludge tanks, to the temperature of the air. This is shown very distinctly in Fig. 15. In this case, the temperatures for one year in a treatment plant at Essen are registered. Intentionally, a plant has been chosen which has to deal with domestic sewage only. In the treatment installations of the Ruhr-Emscher District, which receive also warm industrial waste, the temperature of the sewage is considerably higher. The plant chosen is of great interest, because it consists of two-story tanks and, in addition, separate digestion chambers. The curves show clearly that the yearly average temperature of the sludge in the two-story tanks is nearly 5° higher than in the separate digestion sludge chambers. Upon ascertaining from the Sierp gas curve (Fig. 14) the quantity of gas produced at these temperatures, it is evident that the digestion compartment of a two-story tank in this plant develops at least twice as much gas as a separate digestion chamber of the same dimensions. In plants with warm industrial waste the difference is much greater. Thus, it is clear why separate sludge tanks must always be of a far larger size than two-story tanks.

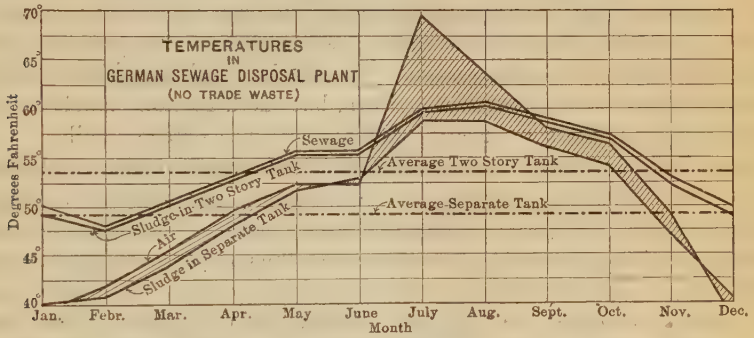

FIG. 15.

At all events, in this manner it is possible to calculate with some certainty from the temperature of the sewage the capacity needed for the digestion chamber of a two-story plant.

Preparatory Treatment of Sewage.—The application of fine screens for preparatory treatment in two-story plants is not clear to the writer from a technical point of view. In Germany, fine screens of good quality are calculated at nearly the same yearly cost (interest, sinking fund, and running expenses) as a complete installation of two-story tanks. The addition of fine screens means the doubling of the yearly expenses without improving the results. The collection and removal of the screenings also is an offensive operation. Thus, the best advantage of a two-story plant has been lost, namely, having sludge that is without odor and at the same time harmless from a sanitary point of view. If, therefore, fine screens are to be arranged in front

of two-story tanks they should be used only for breaking up the scum before it is introduced in sewage.

Tank Design.—There is no doubt whatever that digestion chambers of great depth do more work than shallow ones of the same size. The author proposes to allow for the smaller depth of the digestion chamber by making the chamber larger in proportion to the expected water content of the sludge. No doubt this proposal will find general approval.

The author also convincingly proves that the digestion basins of the two plants at Plainfield, N. J., and Schenectady, N. Y., cannot be used to their full advantage, because of the impossibility of distributing the sludge uniformly among the different sections. When judging these two plants to-day, one must go back to the time when the installations were designed. There are great differences between the fundamental principles of these plants and of the approved German construction of the same time; but it is willingly admitted that at that time it was impossible to foresee the effects of alterations; the plant at Schenectady—as far as is known to the writer—was originally designed for a much greater depth and was only altered during construction on account of the many difficulties that arose.

At the times when these four American plants were designed, German engineers were also of the opinion that the digestion chamber must be arranged with a very large surface. In the meantime this has been proved to be erroneous. Digestion chambers having very small gas vents show the greatest security against foaming and floating sludge. The settling tank should have as large a surface as possible, and as small a depth, but the digestion chambers, on the contrary, should be built as deep as possible with no surface at all.

The recently developed methods of utilizing the gases render the formation of even a small amount of scum very troublesome. Therefore, the construction shown in Fig. 16 is used in the new German plants. The digestion chamber is completely closed on top by a wooden cover, thus keeping the floating sludge entirely under water and allowing only the gases to escape. Below the wooden cover there is arranged, laterally, an opening for the escape of the floating sludge, which flows off in the same manner as the bottom sludge. The wooden cover can never become clogged because under it there is always a layer of gas, keeping off the sludge. The cover consists of a number of wooden laths put together by means of tongue and groove joints. The gases escape through the cracks of the wood. By this arrangement all fears as to scum and foaming are removed. The writer lays special stress on the fact that in the forty plants of the Ruhr-Emscher District foaming is unknown.

Operation.—A fundamental axiom is laid down by Mr. Eddy in the following words:

"The drawing of sludge in the spring should be postponed as long as practicable in order to reduce the danger of removing offensive smelling sludge. On the other hand, it must not be allowed to accumulate to the extent of causing foaming or other difficulties."

These dicta refer only to the northern parts of the United States where the four plants examined by the author are situated. Evidently, the winter there is

so cold that the temperature in the digestion chambers falls below the admissible limit (about 45° Fahr.). In Germany, and likewise in the southern parts of the United States, the conditions are far more favorable. As shown in Fig. 15, the temperature of the sludge in the most unfavorable two-story plant of the Ruhr-Emscher District, never falls below 48° Fahr. In the installations of the Ruhr-Emscher District a good sludge is also obtained during the winter, except that the digestion during the cold season takes more time. In the autumn the surface of the sludge should therefore lie deep, in order to be able to rise slowly during the winter.

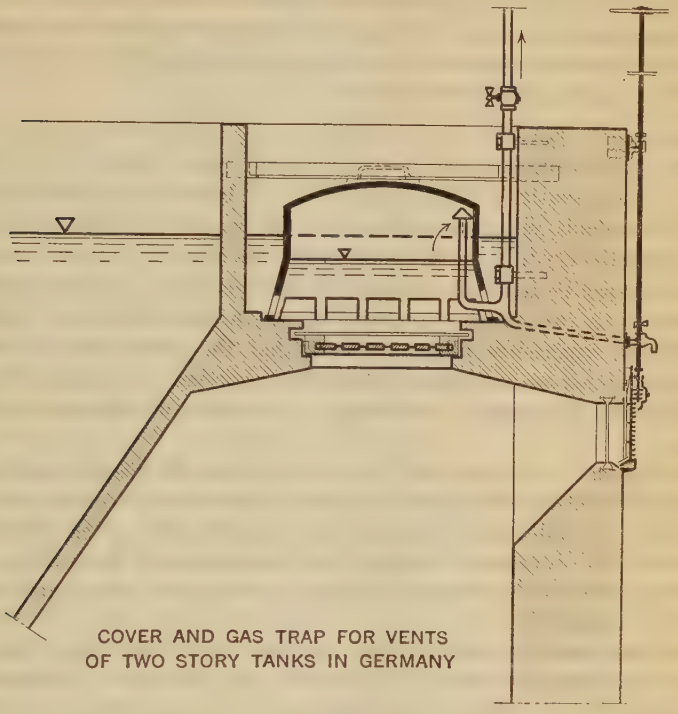

FIG. 16.

It is of great importance to determine what should be done with an overloaded plant during operation. The engineers of the Ruhr-Emscher District have had a great deal of experience in this connection—the oldest plant here having been in operation more than sixteen years—and in a great many of them the quantity of sludge has been increased many-fold. There are the following remedies:

1.—Draw off the sludge frequently, in order to make good use of the digestion chambers.

2.—Enlarge the sludge drying beds, in order to dry even sludge of bad quality.

3.—Arrange sludge lagoons for the winter, as is the practice at some American plants.

4.—Construct secondary digestion chambers, in which the sludge, pumped from the two-story plants, may be stored for some time before it is placed on the sludge drying beds. Such tanks are built in England (Birmingham) with earth embankments.

The last mentioned remedy may be the best solution, as in the case of Schenectady. The sludge can be drawn so often from the different sections of the two-story plant that difficulties arising from scum or foaming cannot occur. From the section at the inlet end relatively fresh sludge will always be drawn, from the central sections only digested sludge. They will be well mixed on their way to the separate digestion chamber, which always operates well as long as the temperature of the air is kept above the admissible limit, namely, 45° Fahr. The digestion chamber must be of such a size, that it can receive the whole of the sludge during the winter.

Harrison P. Eddy,* M. Am. Soc. C. E. (by letter).—The writer is gratified by the valuable discussion which has been brought out by the comparison of the essential conditions affecting the behavior of Imhoff tanks.

Mr. Hammond's suggestion, that it would be wise to instruct the operator to determine the capacity of the tanks under his care and operate them according to the capacity found by test, is excellent advice. Usually, however, the operator has to run the tanks according to the load delivered to the plant and although experience might indicate that the tanks were of insufficient capacity, he must operate them under the overload, or by-pass the sewage. The by-passing of sewage is, of course, a procedure which cannot be done in all cases.

Mr. Hammond's theorem that "the tank having the greatest ratio of depth to capacity would give the most successful performance" seems to be just the converse of that indicated by experience. According to his theorem the tanks at Plainfield, N. J., should function more satisfactorily than those at Fitchburg, Mass., the ratios of depths to cubic foot capacity per capita being 13.25 and 13.12, respectively.

Experience seems to indicate that, all other conditions being identical, the following theorem may more nearly approximate the facts: "The tank having the greatest ratio of depth to number of persons served per cubic foot of sludge compartment capacity will function the most successfully." If this theorem is correct, it should follow that a certain ratio will give satisfactory results for the several depths, at least within limited ranges. The application of this theorem to tanks 10, 20, and 30 ft. deep, on the assumption that 40 is the optimum ratio, gives the results shown in Table 22.

The tendency indicated by Table 22 appears to be borne out by experience, although it should be noted that this theorem means simply that, other conditions being the same, sludge compartment capacity expressed in persons served per cubic foot is directly proportional to depth.

The depth of Imhoff tanks appears to have relatively little effect on temperature conditions in the digestion compartment, as this temperature follows very closely the temperatures of the sewage delivered to the tanks. The tem-

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

perature changes which take place in the sedimentation compartment and in the sludge digestion compartment are not particularly rapid in either. Although the temperatures of the sewage and sludge in a shallow tank would tend to respond somewhat more quickly to changes in atmospheric temperature, the actual temperature change in sewage and sludge would be relatively small and unimportant. It is quite likely that daily changes in barometric pressure are of more immediate effect on foaming or the rising of gas-lifted sludge than daily changes in temperature.

TABLE 22.

Depth of tanks, in feet.	Optimum ratio of depth to number of persons per cubic foot capacity.	Sludge Compartment Capacity.	
		Persons per cubic foot.	Cubic feet per capita.
10	40	0.25	4
20	40	0.50	2
30	40	0.75	1.33

For the sake of accuracy, it should be stated that free interconnection of sludge digestion compartments has been put into effect in only one of the nine tanks at Schenectady, N. Y. In this one tank the original openings in the partition walls were simply extended to the bottom of the tank, the net effect of the changes resulting in far less ease of intercommunication than in the case of the plants at Fitchburg, or Rochester, N. Y. Lime has not been used at Schenectady for several years.

Such foaming as has occurred at Fitchburg in the period of ten years since operation of the plant was started, has been inconsequential in amount compared with foaming difficulties in Plainfield or Schenectady.

The greater proportion of mineral matter present in the bottom sludge as drawn than in the suspended solids of the sewage or in the scum is explained by Mr. Skinner as being due to the concentration of particles of mineral matter in the bottom of the tank. Another reason is the actual gasification and liquefaction of the organic portion of the sludge particles which, in the older and deeper lying sludge, will have proceeded to a greater extent than in the upper and more recently deposited sludge.

The arrangement of the tanks at Schenectady with eight practically separate sludge compartments in a line results in a tendency for the end compartments to become overloaded, although unsatisfactory conditions with respect to foaming and rising scum have occurred in the central as well as in the end compartments.

The writer does not feel that the hopper bottom has any advantage over the trough bottom, providing the sludge draw-off pipes are equally distant from each other in both cases. In fact, under some conditions, there may be an advantage in the trough bottom. A large proportion of the bottom provided with hoppers is filled with the mass of masonry between the apices of adjoining hoppers. In trough bottoms this space may be occupied with

mineral or other inert matter deposited from the sewage, but there does not appear to be any reason why the volume of such deposit should be greater than that of the masonry of the hopper bottom. In the absence of considerable grit the space thus occupied may be much less than that occupied by the masonry. Sludge will flow practically as well over the deposits as over the concrete hoppers. With trough bottoms, sludge draw-off pipes may be placed at shorter intervals, if desired, than in the case of hopper bottoms, thus making available space which otherwise would be occupied by deposits in the case of troughs or by masonry in the case of hoppers.

The writer is not sure that tall narrow gas vents are other than a secondary factor in the foaming phenomena. Foaming appears to be connected with a very rapid formation of gas, a biological condition which must be due primarily to factors affecting micro-organic activity. It is conceivable that where such rapid formation of gas is produced, the tall vents may result in the lifting of the volume of liquid in the gas vents similar to the raising of water by air lifts, as suggested by Mr. Skinner, although it is not quite clear from his discussion how the sealing of the slot by the rising sludge will produce a condition favorable to foaming from the standpoint of air lift.

The writer is unable to agree with Mr. Skinner's method for computing the capacity of sludge digestion compartments. Although this method takes into account certain variables, it does not give any weight to the effect of depth of tank which the writer believes to be important. Thus, by Mr. Skinner's method, the computed volume of sludge compartment capacity would be the same for a tank 10 ft. deep as for one 30 ft. deep. This does not seem logical when conditions within the sludge compartment are taken into account. A rational method would be to base the volume on an estimate of the density of sludge which appears to vary with the depth of the tank.

It is possible, of course, to compute the capacity of any sludge compartment in terms of cubic feet per capita, but as a means of computing the required capacity the writer does not think it is safe to provide as uniform a number of cubic feet per capita in all cases. Such a uniform allowance, however, may be made where all conditions except tributary population are identical.

Dr. Imhoff's rule of 1.2 cu. ft. capacity for sewage from separate sewers cannot be applied equally well to tanks in the South and in the North and to compartments receiving 50% and 75%, respectively, of the suspended solids of the sewage. Neither does the writer think that the same arbitrary capacity per capita can be applied to tanks of, say, 15 ft. depth and 30 ft. depth.

The formula presented by Mr. Skinner takes into account several of the variables; special allowances for others can be made, or the suggested factor of safety can be applied to cover them. In this formula, he has assumed a sludge containing 90% of water in the tank and has allowed 25% reduction for the combined effect of concentration and digestion. The density of the sludge, as already pointed out, will vary with the depth. Sludge containing approximately 85% moisture has frequently been drawn at Fitchburg; the average digestion during the year approximates 30 per cent. The allowances for density of sludge and for the digestion accomplished must be based on

the design of the tank (depth being the major controlling factor), on temperature, and on the composition of the sewage.

The aids to proper functioning suggested by Mr. Skinner are excellent, although there is some question in the writer's mind as to the necessity for wide gas vents.

Mr. Downes has directed attention to the importance of adequate provision for sludge dewatering facilities so that the sludge may be drawn from the tanks as soon as it is ready. A moment's consideration is sufficient to demonstrate that it is uneconomical to delay the efficient functioning of tanks because of inadequate sludge dewatering facilities. The cost of the latter is much less than that of tanks.

The use of alum for conditioning sludge of poor quality appears to have been a valuable aid in the case of the Plainfield plant, but similar satisfactory results have not been obtained at Schenectady.

It should be pointed out that the writer, in computing Tables 7 and 8, was basing his computations on purely arbitrary assumptions and the tables were set up for illustrative purposes only. As a matter of fact, deep tanks may be expected to contain sludge which will reach a density of 15% solids at the bottom. At Plainfield and Schenectady, however, with shallow tanks, it is not to be expected that sludges of such density can be obtained.

Mr. Downes' explanation and illustration of the gas action in the Fitchburg and Plainfield tanks is interesting and has much merit. In the deeper tanks there is not only a greater accumulation of sludge in the hoppers per unit area of sludge surface, and, consequently, a greater number of sludge bubbles given off per unit of area, but there is a greater compression of the gases. It is likely that the larger number and greater expansion of rising gas bubbles in the deeper tanks play an important part in the agitation of the liquid and sludge, thereby assisting favorable digestion processes in the digestion and scum compartments. Furthermore, due to the greater pressure of water in the bottom of deep tanks where gas is being produced, the gases do not acquire sufficient volume to lift the sludge particles, until opportunity has been given for material digestion of the sludge. This digestion renders the particles and the mass of sludge more easily disrupted by the explosive force of the expanding gases as the sludge with the entrained gases rises toward the surface. Due to this disruption, the particles of solid matter freed from gas sink back to the bottom of the tank instead of rising to the surface in the form of floating scum or foam.

The writer is not sure that Imhoff tanks which require for the breaking up of scum in the gas vents more labor than may be necessary at other plants, or the use of mechanical agitators, are unsatisfactory installations. If such measures are successful in producing satisfactory digested sludge, their use is justified, provided the costs are not unduly high.

Unquestionably the slot must be placed sufficiently high to permit the accumulation of sludge in quantities estimated to occur in the time interval required for satisfactory digestion, the controlling time being that of the winter months of low temperature when digestion proceeds at a slow rate. It

seems desirable to have in addition a substantial vertical distance to give a margin of safety, the so-called "neutral zone", on which the sludge accumulations will not encroach.

Mr. Downes is correct in his statement that the percentage of solids is not sufficient to express the true relation of solids content to volume of sludge. Specific gravity also is required to give the correct relation. For general purposes, however, as far as the sludge lying in the tank is concerned, it is probably within the limits of error to consider that the volume of sludge per pound of dry solids content varies in inverse proportion to the percentage of solids. It is evident, of course, that the specific gravity of the sludge lying in the tanks must be greater than 1.0 and that the dissolved gases in the sludge have little effect on its specific gravity, except when the pressure of the supernatant liquid is released or reduced. Free gas bubbles reduce the specific gravity of the sludge to 1.0 or less, and in the latter case the sludge rises as scum or foam. When it comes to that portion of the sludge which has become well digested and is drawn from the tanks, or to dewatered sludge lying on sludge beds, the same relation does not hold true due to the interfering effect of voids resulting from expansion of gases and to the effect of changes in character of the solids comprising the sludge.

The drawing of sludge as early in the year as inoffensive sludge can be obtained is an excellent practice as emphasized by Mr. Hartwell. This course of action provides the maximum space in the digestion compartment during the summer period of intense activity and gas production, when foaming is most likely to occur.

The troubles from oil and grease in Imhoff tanks, referred to by Mr. Cohn, have been experienced at various other plants. In the design for the new sewage treatment works at Akron, Ohio, provision has been made for a grease-skimming tank preceding the Imhoff tanks for the interception of such grease and oil. It does not seem probable that variations in the quantity of soap curd in different sewages play more than a minor part in the functioning of Imhoff tanks. The spent mash from the illicit manufacture of home brew may tend to produce a large amount of scum difficult to handle, in which biological action is mainly fermentation with the production of carbon dioxide. With excessive production of carbon dioxide, there may actually occur an inhibition of the other biological activities which produce proper digestion, as indicated by the production of methane.

The hydrogen-ion concentration results thus far obtained at Schenectady appear to offer little in the way of explanation for the conditions found to prevail there. The temperature records show that, as previously pointed out by the writer, the sludge compartment has approximately the same temperatures as the sewage and tank effluent.

If the use of mechanical agitators, as described by Mr. Learned, permit the reduction of the required capacity of sludge digestion compartments from 2 cu. ft. or more per capita to 0.57 cu. ft., or 1.0 cu. ft. per capita, as at Newton, Kans., and Enid, Okla., respectively, such equipment might well be a desirable feature. The costs for such equipment and its operation do not

appear to be excessive, although complete data in this connection are not given by Mr. Learned.

Under identical conditions the tank that will function properly without resort to the breaking up of the scum in the gas vents by hosing or other laborious methods and without mechanical agitation, is more satisfactory than the tank which requires this excessive labor or mechanical devices. If by skillful design proper functioning can be secured this should be done.

It is interesting that the rise of the sludge in the plant at Broken Bow Okla., occurred at approximately the same season in the spring as the semi-annual overturn in water reservoirs. In both cases, changes in specific gravity in the top and bottom layers are responsible. In the case of water reservoirs the change in specific gravity is due directly to differences in temperature between the bottom and top layers, whereas in the case of Imhoff tanks the action of the rising temperature in the Spring is to stimulate biological activity and the production of gas, thereby producing sludge of specific gravity less than that of water.

Dr. Imhoff's diagram (Fig. 14) comparing the evolution of gas from sludge at Essen, Germany, in 1924, with that at Worcester, Mass., in 1902, shows comparatively close agreement.

American data substantiate Dr. Imhoff's statement to the effect that the temperatures in the sludge digestion compartment of Imhoff tanks follow those of the sewage. There are, however, no American data, as far as the writer knows, to show the temperature of sludge in separate digestion tanks.

The difference between temperature in separate sludge digestion tanks and in Imhoff tanks depends on climatic conditions and tank construction. The covering of separate sludge digestion tanks would tend toward as high temperature in separate digestion tanks as in Imhoff tanks. There is no justification to assume without qualification that separate sludge digestion tanks will have a much lower temperature than Imhoff tanks.

The cover and gas-trap for the vents of two-story tanks as used by Dr. Imhoff in Germany are of much interest. It is not quite clear how the gases can escape through the cracks of the wood and at the same time persist below the cover to such an extent as to prevent the sludge rising to the under side of the wooden cover. It would appear possible for grease and sludge particles to clog the cracks in the wooden cover and to seal the vent completely, unless a cushion of gas were present below the cover.

Dr. Imhoff's suggestion for constructing separate sludge digestion tanks to provide additional digestion capacity at those installations of Imhoff tanks where experience has shown such capacity to be inadequate, points the way for remedying conditions where other methods fail.

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Paper No. 1561

WATERWAY AND RAILWAY EQUIVALENTS*

By William M. Black,† M. Am. Soc. C. E.

With Discussion by Messrs. C. E. Grunsky, C. H. Markham, Edward Burr, William G. Atwood, J. R. Slattery, C. I. Grimm, H. F. Dunham, William H. Burr, and William M. Black.

Synopsis

New waterway lines of transportation should not be established unless an analysis of the existing railway, highway, and waterway lines and of the need for transportation shows that the proposed new line is required and that if established it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs.

As the cost of all lines of communication is paid for from the public wealth, an exercise of unprejudiced judgment is demanded before an investment is made. In selecting any one of the general transportation agencies for use, the total tonnage requiring movement must be analyzed as to volume, distance to be moved, continuity of traffic, kinds of commodities, importance of time of transit, class of carriers required, and terminal facilities.

The cost of transportation by any method depends primarily on the cost of moving the required tonnage. In this cost must enter the charges for the interest on the capital cost of the carriers and of the roadbed or waterway, for the amortization of the investment, and the cost of maintenance and operation. In this last cost, the proportion due to the time element in operation must be included.

The cost of movement along a waterway is a function of the resistance offered by the water to the movement of a boat; such resistance varies with

* Presented at the meeting of December 10, 1924.

† Maj.-Gen., U. S. A. (*Retired*); Cons. Engr. (Black, McKenney & Stewart), Washington, D. C.

the speed, with the surface and form of the boat, and with the dimensions of the waterway. The resistance increases much more rapidly than the speed, especially in channels of restricted dimensions.

In forming a project for a waterway, the size and speed for the carriers necessary to meet traffic conditions should first be determined, and from these the dimensions necessary for an adequate waterway. Many experiments have been made in Europe and America in order to determine the resistance to movement of boats of various classes in channels of different dimensions. The formulas deduced are empirical and must be applied with judgment. A number of references are given to authorities on this subject, to de Mas, Grübe, and Haack in Europe, and to Sweet, Taylor, Sadler, and McEntee in America. Probably the most comprehensive series of experiments were those made under the direction of the Engineer Department, U. S. Army, by the United States Tow Boat Board. The Government publications on this subject are noted; also the conclusions reached. When a speed exceeding 3 miles per hour is necessary, the form of the boat becomes important, as shown by examples. From the U. S. Navy tank tests, it appears that if, for traffic reasons, a cargo carrier with a load capacity of 2 000 tons and a speed of 5 miles per hour is necessary, the minimum channel dimensions should be a depth of 10 ft. and a bottom width of 188 ft.

The writer then discusses the increase in depth required to allow for the squat of boats, and the increases in channel width required at bends and to allow for the oblique position taken by boats and tows subjected to a heavy side wind. Provision must be made to prevent the erosion of banks under the action of waves due to the boat and in restricted waterways due to current velocities actuated under and past the hull.

In the Mississippi Valley waterways, tows are generally pushed by stern-wheel or tunnel towboats; in the Hudson River System, they are pulled by propellers. Tests on the Mississippi River showed that for six barges a formation of two abreast and three deep offered the least resistance.

In studying a project for a waterway all elements entering into costs of transportation must be considered and savings in transportation costs must be weighed against these elements.

Statistics are cited to show that under comparable conditions the power required for propulsion is 3.13 lb. per ton of train weight for railways as contrasted with 1.87 lb. per ton of displacement for waterways; as to carrying capacity during 24 hours, the relation is 144 000 tons of freight for a single-track railway to 884 000 tons for the waterway.

Undoubtedly, transportation is cheaper by adequate waterways than by rail for long runs when the time element is not of primary importance, and quicker as well as cheaper for comparatively short runs. Too little attention, however, has been given in the past to the construction of waterways with reference to the class of carriers required and to their equipment with the most suitable carriers. The relative advantages and relative costs of each medium of transportation should be weighed with care before new projects are undertaken. Destructive competition should cease; co-operation should be required.

Before this country was developed, any improvement of its transportation facilities was advantageous: The opening of a dirt road to take the place of a trail was desirable; the clearing of a watercourse, or the construction of a canal of any navigable kind, formed a beneficial public improvement. To-day, however, almost all sections are provided with transportation facilities such as were beyond the imagination of the Colonists so that when new lines of transportation are created, the money and labor expended are wasted unless they are needed by commerce to provide a means of transportation better and at lower cost than the facilities which already exist. Railways with their continually improving equipment, and automobiles traveling on the smooth, hard roads, carry freight more cheaply than land carriers elsewhere in the world. On the other hand, the general use of the internal waterways as freight carriers has diminished. To-day, if a waterway will not accommodate carriers which in safety and cost of operation are superior to the land carriers, the waterway is not used. The old primary importance of internal waterways as a means of transportation has vanished. The expediency of their use for this purpose is now relative, and such use is based on the economy it will produce.

It is a National policy to create and maintain highways and waterway channels at the public expense. This expense, like the cost of railways, must be defrayed from the general wealth of the people. Before new projects for highway or waterway construction are entered into the same care should be exercised as is used before private funds are invested. The cost of the undertaking must be estimated, the facilities it will create must be determined, and the economies which these facilities will produce must be weighed against the legitimate interest charges on the investment and the cost of maintenance and operation of the work.

In the case of a proposed creation of a new line of communication by water, the first thing to be considered is the need—the traffic that will use it and the decrease in the cost of transportation such use will produce. In considering the traffic, its component commodities must be detailed, also their origin and destination, the importance of the time of transit, the class of carrier most suitable, the number of such carriers required, and the best and most economical means for their propulsion. Consideration must be given to the continuity of the traffic and as to whether it originates from few or from many sources, whether it is destined for few or for many consignees, and whether it is to be carried in bulk or in packages. In considering the probable diminution of the cost of transportation, there must be borne in mind the interest on the capital cost of the new channel and the cost of its maintenance, the capital costs of towboats and barges and of their maintenance, the cost of operation, including that of loading and unloading, terminal charges, and the charges for time laid up, etc., and, in addition, the amortization of the investment.

It is evident that even when there is a real commercial demand for a waterway, such a line of communication will not be a financial success unless the channel available is suitable for navigation by the class of carriers required, and unless the channel facilities permit an inexpensive type of carrier.

Water Resistances.—The cost of movement depends on the resistance offered by the water, this being a function of the surface, form, and speed of the carrier, and depth and width of the channel. The resistance of the water, and, consequently, the power required to move the carrier, increases rapidly with the speed, somewhat more than in the proportion of the square of the speed. This is important, since with high wages and high cost of subsistence the time consumed in a voyage forms an important element in operating costs. The increase of power required to produce an increase of speed in shoal water is exemplified by an experiment made in 1911 with the steamer, *Hendrik Hudson,* which is 400 ft. long, with a beam of 45 ft. and a draft of 9½ to 10 ft. Moving in a channel 14 ft. deep and about 900 ft. wide, she attained a speed of 9.1 miles per hour with the exertion of 350 h.p., while to attain a speed of 9.48 miles per hour, 730 h.p. was required.

Toward the close of the Nineteenth Century extended experiments were made in this country and in Europe to determine the resistance developed by boats moving in restricted channels. In the United States, the late Elnathan Sweet, M. Am. Soc. C. E., made experiments on the Erie Canal and proposed a formula for resistance which has been published.* In 1901, Mr. Sweet again made an analysis of the subject† and, in 1902, wrote another paper‡ in which he repeated a part of his report on the Barge Canal. He invited particular attention to the experiments made in Germany by R. Haack, who evolved an interesting method of determining resistance to traction in canals from the differences in canal level in front of and behind a boat. Mr. Sweet mentioned the important effect of these differences of level in producing velocities in the water along the sides and bottom of the canal with consequent erosive action.

Channel Dimensions.—Haack's formula was used in fixing the dimensions of the Panama Canal. The method of application in that instance, together with two plates showing, respectively, the curve of depression of water surface for varying ratios of ship and canal cross-sections and the speed curves for restricted channels, have been published.§ Later, Professor Sadler, of the University of Michigan, and Admiral Taylor and other officers of the U. S. Navy, made many tests of model boats in tanks and deduced useful formulas. Other tests with models and with full-sized carriers were made under the direction of the U. S. Engineer Department by the U. S. Tow Boat Board.‖ A report of model experiments with canal barges in canals of varying sizes was made in 1917 and 1918, by Constructor William McEntee, U. S. N., and a discussion of the subject by Colonel F. R. Shunk, Corps of Engineers, U. S. A., has been published by the War Department.¶ Valuable data showing the application of the deductions of Professor Sadler and others to the case of the channel of the Upper Hudson River have also been pub-

* *Transactions,* Am. Soc. C. E., Vol. IX (1880), p. 99.

† Report on Barge Canal, State of New York, 1901, p. 599.

‡ "Some Important Phases of Canal Navigation, Illustrated by Recent Experiments in Germany," *Transactions,* Am. Soc. C. E., Vol. XLVII (1902), p. 435.

§ "The Panama Canal," Pt. I, p. 69, Govt. Printing Office, 1905.

‖ The reports of this Board are published in H. R. Doc. 857, 63d Cong., 2d Sess., and H. R. Doc. 108, 67th Cong., 1st Sess.

¶ "Reports on Model Tests of Barges in Waterways of Restricted Dimensions," 1918.

lished by the Government.* The conclusions reached from the model tests agree fairly well with the results of the tests with full-sized boats, and these should be a guide for engineers in planning new waterways, as well as for naval architects engaged in the design of boats.

A full discussion of this difficult subject would be too extensive; some of the conclusions reached, however, can be given.

The best dimensions for channels, from the standpoint of forming commercial lanes, depend on the form, dimensions, and speeds required for the carriers. These, in turn, depend on the nature of the commercial movement. In the case of natural waterways it may be found that adequate channels cannot be formed, or that the cost of the required improvement would be excessive. In the case of canals, except for the supply of water for the summit level, cost alone limits the dimensions to be given. In either case if interest on the cost of the improvement plus the cost of its maintenance exceeds, or even equals, the demonstrable saving in transportation costs, the improvement should not be made.

So many varying conditions exist as to the width, depth, and constitution of the bed of navigable streams, and to a certain extent also of artificial waterways, that the formulas for channel and carrier dimensions and forms are empirical. In general, it may be said that in considering a project for a waterway to serve a certain volume and class of traffic, the economic type of carrier, its capacity, and required speed of movement should first be determined. Next, the dimensions required for the channel should be fixed. Again, it should be remembered that if the requirements thus determined make the proposed waterway too costly, the project should be abandoned.

Type of Carrier.—In selecting the type of carrier some of the governing principles are, as follows: As each carrier is only a trussed body, the general principle applies that, other things being equal, the deeper the truss the stronger it will be for the same weight of material. Increase of draft tends toward economy of construction.

When rapidity of transit is important, single, self-propelled carriers, with moulded hull, are best. When it is desirable to move large volumes of freight at a minimum cost, towed barges are more economical, provided the available channel has sufficient width and the bends have sufficient radius. The towing system also makes it possible to keep the expensive towboat in use while the separate carriers are waiting for loading or discharging. As a result of tests on the Mississippi the Experimental Tow Boat Board reported as follows:† "In going ahead, the resistance per ton of displacement decreases as the number of barges increases, if the formation is kept narrow."

Type of Towboat.—The U. S. Tow Boat Board made many experiments as to the best type of towboat for use on the Mississippi River, and its conclusions and the method of attaining them are available.‡ For the Mississippi River, the Board recommended for towboats either steel, stern-wheel steamers

* Appendix G of H. R. Doc. 1160, 63d Cong., 3d Sess.
† H. R. Doc. 108, 67th Cong., 1st Sess., p. 13.
‡ *Loc. cit.*, pp. 10–12, 15.

of 4½-ft. draft, with 1 800-h.p. engines, or 1 800-h.p., steel, screw-tunnel boats of 7-ft. draft.

A discussion on towboats for the Hudson River has also been published by the Government* in which it is stated:

"Experience has shown that from 13 to 15 ft. is the most economical draft for towboats of the type used on the lower Hudson and in New York Harbor * * *. Based on the best boats now in service on the Hudson, the following seems to be the limiting indicated horse-powers practicable for various draft tugs:

"Draft.	I. H. P.
6 to 8 ft.	150 to 300
8.5 to 10 ft.	500 to 800
13 to 15 ft.	1 000 to 1 800

With equal indicated horse-powers, the pulling power generally increases with the draft. The smallest tugs in service are operated with a crew of 9 men, and the largest with a crew of 12 men. For a given total pulling power, however, the coal consumption varies more nearly with the total horse-power required, however distributed between tugs. The cost of wages and repairs is more nearly a function of the number of boats. These figures give some idea of the economy in using deep draft boats.

"It has been found, by experiment and experience, that the standard formulas for resistance to motion in general use in marine architecture do not give reliable results for the channel conditions prevailing on the upper Hudson. The best available guide is believed to be contained in curves deduced by Professor Sadler as a result of his model-tank experiments referred to above and are presented in an article entitled 'The Resistance of Some Merchant Ship Types in Shallow Water', read at the Eighteenth General Meeting of the Society of Naval Architects and Marine Engineers, on November 16 and 17, 1911. Based on his curves for 'full type of cargo boat', the following table [Table 1] of resistances and effective horse-powers were computed for a barge 270 ft. long and 40 ft. beam, drawing 9.5 ft. The effective horse-power is that computed as necessary to exert the towline resistances given."

TABLE 1.—Relation of Towline Tension and Power Required, for Various Speeds and Drafts.

Speed:		Towline Tension, in Tons.			Effective Horse-Power Required to Overcome Towline Tension = ½ I. H. P.		
In miles per hour.	In feet per second.	D = 12 ft.	D = 14 ft.	D = 16 ft.	D = 12 ft.	D = 14 ft.	D = 16 ft.
4	5.866	1.35	0.75	0.66	28.8	25.7	23.5
6	8.8	2.1	1.5	1.35	67.5	48.0	43.0
8	11.74	5.25	3.0	2.55	251.4	128.1	108.86

Tows on the Mississippi River are pushed by towboats, steering being accomplished by the towboat rudders and by the propelling wheel or screws. Use is also made of the pressure of the current on the side of the tow. Much of the time, particularly in down-stream movements, the towboat wheel is backing in order to throw the head of the tow around—this operation is termed

* H. R. Doc. 1160, 63d Cong., 3d Sess., p. 103.

"flanking". On the Hudson, tows are hauled on a line. This difference in practice causes a marked difference in the class of towboats used.

Type of Barges and Barge Resistances.—The type of hull to be selected for barges is largely dependent on the relative importance of carrying capacity and of water resistance. In still water, at low speeds and in channels of suitable dimensions, the relative importance of the form of hull decreases; but in natural waterways, as the resistance to movement increases very rapidly with an increase of speed, and as the velocity of the boat with reference to the water must be sufficient to afford a suitable rate of progress when moving against a current, the form of the barge becomes of primary importance.

After numerous experiments in the canalized section of the Seine River, M. de Mas, Inspecteur Général des Ponts et Chaussées, proposed* as the best type two barge forms, each 126.7 ft. long by 16.4 ft. wide by 7 ft. deep (length-width ratio = 7.7), having a spoon-shaped bow and stern, but with rakes of different lengths, 14.8 ft. at each end for one, and 8.2 ft. for the other. He states that the resistance offered by the water to the movement of a boat for all speeds greater than 1.7 miles per hour, can be expressed by the formula, $r = (a + b\,t)\ V^{2.25}$, in which, r is the resistance, in kilogrammes; a and b are constants varying with the form of barge; t is the draft, in meters; and V is the speed, in meters per second.

M. de Mas deduced the following constants for barges of varying forms but of similar dimensions, 38.5 by 5 by 1.9 m.:†

Péniche	$a = 21.3$	$b = 123.6$
Flûte	$a = 21.5$	$b = 78.1$
Toue	$a = 14.2$	$b = 52.4$

In comparison, the models he recommends, at a speed of 1.5 m. per sec., offer a resistance only two-thirds as great as the flûtes. The experiments covered only a limited range as to boats, speed, and channel dimensions, and the conclusions are valid only within that range.

After many experiments, the U. S. Tow Boat Board recommended the construction of steel barges for the Mississippi River freight lines on a model quite similar to those recommended by de Mas. This type of barge is 300 ft. long by 48 ft. wide by 10 ft. deep (length-width ratio = 6.2). It has a modified spoon-shaped bow and stern, 30 ft. wide at the top, with a rake 42 ft. long, and has rounded bilges of 5-ft. radius. At 6-ft. draft, its load is 1 660 short tons; at 8-ft. draft, 2 500 short tons; and at 9-ft. draft, 3 000 short tons. In practical operation this form of scow has shown a resistance markedly less than the usual type. The resistance per short ton of displacement developed by a tow of four of these barges, arranged two abreast, at a speed of 5½ miles per hour, was 2.15 lb., while a similarly arranged tow of barges of the usual scow shape, moving at the same speed, developed a resistance of 3.6 lb. per ton of displacement.

* "Rivères à Courant Libre", 1899 Edition.

† The péniche is a scow with slightly rounded ends and a small rake. The flûte is a scow with ends similar to the usual canal-boat form. The toue is also similar to the canal-boat form, but with a longer rake at the bow.

The U. S. Tow Boat Board in its discussion of the resistances to motion of barges* shows that the economical speed for up-stream towing is 50% greater than the velocity of the stream; the Board states:

"The resistances to the motion of a boat are classified as frictional and residual, or wave making. The former is dependent upon the extent and character of the wetted surface of the boat and the speed of movement, and the latter upon the form of vessel, the depth of water and the speed."

The Board quotes Froude's formula for the frictional resistance, namely, $R_f = F\ S\ V^{1.83}$, in which, R represents the frictional resistance; F, Tideman's constant = 0.0095, which is the frictional resistance, in pounds per square foot at a speed of 1 knot; V, the speed, in knots; and S, the wetted surface, in square feet. The Board also quotes Taylor's formula giving the minimum depth of water required in order that the resistance may not increase with increased speed, namely, depth $= 10\ H \frac{V}{\sqrt{L}}$, in which, H = draft, in feet; V = speed, in knots; and L = length of water line, in feet.

The Board made experiments in a broad waterway to determine resistances, with results shown in Table 2; in a narrow waterway the friction on the banks would enter to a greater extent. It will be noted that the form (or wave-making) resistance is more than half the total and that proportionally the form resistance increases rapidly as the depth of water decreases.

TABLE 2.—Comparison of Resistances to a Scow-Shaped Barge for Different Depths of Water.

	In deep water.	In 12 ft. of water.	In 9 ft. of water.
Total resistance (by experiment)	2 371	3 250	4 350
Frictional resistance (by formula)	997	997	997
Residual resistance (difference between above)	1 374	2 253	3 353
Portion of resistance due to surface, percentage	42	30.7	22.9
" " " " " form, percentage	58	69.3	77.1

Constructor McEntee states:

"The resistance expressed as a percentage of the displacement increases very quickly with the speed. For the best form of barge† in a canal of moderate size, the speed in miles per hour cannot exceed 0.55 $\sqrt{L}$ (L = length of barge, in feet) without increasing the resistance beyond one per cent. of the displacement * * *. The draft of the barge should be about half the depth of water in the canal, but it can be increased to three-fourths of the depth of water without greatly increasing the resistance providing the sectional area ratio (ratio between the area of maximum wetted cross-section of barge and area of cross-section of the canal) is not changed. With a given cross-sectional area of barge, increase in length reduces the resistance per ton at the same speed. For a barge of length seven times its breadth an increase of 30 per cent. in length will reduce the resistance per ton about 10 per cent. to 15 per cent."

* H. R. Doc. 857, 63d Cong., 2d Sess., pp. 6–7.

† Constructor McEntee's experiments were made mainly with models in which the length-beam ratio was 7, the ratio found to be the most advisable in earlier tests.

The best type of barge for use under given circumstances is so dependent on the condition of the waterway in which it is to be navigated that the state of the channel must be known before a selection can be made.

Channel Dimensions.—This subject has been considered in several of the sessions of the International Congress of Navigation, and in the reports of the Congress are given the practice in Europe and the conclusions of European waterway experts. In studying European practice, however, it should be borne in mind that with their low standard of wages and low cost of living, the time element, which comprises at least 50% of all the movement costs, permits of the use of speeds which in the United States would be too slow to be economical. In Europe, too, the higher cost of movement by rail allows a margin for increased costs by water which is not found in the United States. Except on rivers like the Rhine and Danube, the barges used on European waterways are too small for economical general use in the United States.

In the European canals, revetment of the banks is general, and frequently a berm is constructed at about the lower limit of destructive wave action. In Russia, this point is found to vary from 13½ to 40 in. below the normal water level. In rock cuts, for one-way traffic, the minimum canal width recommended is 1.4 times the width of the largest boat. In order to allow boats to pass each other, this width should be increased to 2.8 times the width of the largest boat. Another rule, for boats of 300 tons, recommends a least width for the area having the maximum depth, of double the greatest width of the boat plus 16.4 ft., increasing this width by 1.64 ft. for each additional 100 tons in size. The side slopes used vary with the nature of the material of the banks. The depths recommended are from 1.5 to 3.28 ft. below the bottom of the deepest draft boat. When towboats are propelled by a screw, a clearance of 2 ft. 6 in. is recommended. European authorities recommend that the ratio between the cross-section of the canal and the cross-section of the immersed part of the boat should be from 3 to 5.

Experimental Results.—The results of the tests of the U. S. Tow Boat Board with models and barges of various sizes and in channels of various dimensions, are shown by curves published in its reports. Constructor McEntee's tests were made with models and in his "Report on Model Tests of Barges", previously mentioned, the curves published show the results obtained by him. He summarizes his conclusions from these tests as follows:

"1.—The resistance expressed as a percentage of the displacement increases very quickly with the speed. For the best form of barge in a canal of moderate size, the speed in miles per hour cannot exceed $0.55\sqrt{L}$ without increasing the resistance beyond 1% of the displacement. In this statement the quantity $\sqrt{L}$ = square root of the barge length in feet. For a 100-ft. barge this limit would be about 5.5 miles per hour.

"2.—After speed the most important element affecting resistance is the ratio of the cross-sectional area of the canal to that of the barge. For a given barge running at a moderate fixed speed as this ratio is increased from 4 to 8, the resistance is reduced as much as 50 per cent.

"3.—The proportions of the canal—that is, the ratio between its width and depth for a given sectional area—is a factor of importance. For canals of varying width-depth ratio, but of sectional area bearing a constant relation to the

sectional area of a given barge, the best results are obtained when the width of the canal at the bottom is about two times the depth of the water. On the other hand, if the beam and length of the barge are increased directly as the width of the canal, the draft being maintained at 75 per cent. of the depth of water, the best width-depth ratio for the canal is about 9 to 11.

"4.—For self-propelled barges, or when towboats are used, the deep and narrow canal will have decided advantages resulting from the possibility of using larger and more efficient propellers than could be used in the shallow and wide canals. This conclusion is based upon estimates, and not on the results of the present experiments except in so far as the resistance of the barges is concerned.

"5.—The draft of the barge should be about half the depth of water in the canal, but it can be increased to three-fourths of the depth of water without greatly increasing the resistance, provided the sectional area ratio is not changed.

"6.—With a given cross-sectional area of barge, increase in length reduces the resistance per ton at the same speed. For a barge of length seven times its breadth an increase of 30 per cent. in length will reduce the resistance per ton about 10 to 15 per cent. The length is limited by considerations of strength, handling, locks, etc., and not by questions of resistance."

As a result of the tests, the conclusion may be reached that, in general, when the waterway is to be used by boats of all sizes, the depth should be 25% greater than the maximum draft of the largest boat, and that the ratio of bottom width of channel to depth should be 11; but if the waterway is to be used only by barges of one size and which have the same length-beam ratio, the resistances are least for a ratio of width to depth of channel of 2, which would make the channel too narrow for two-way traffic. Therefore, the minimum width of channel is considered to be 2.5 times the beam of the largest boat using it. From Constructor McEntee's curves it is evident that, when the time element is of primary importance or when by reason of a current speed is essential, the cross-section of the canal must be increased to as great an extent as would be warranted by the commercial value of the waterway. In commenting on Colonel Shunk's discussion previously referred to, the Acting Chief of Engineers states:

"* * * the apparent advantage of increasing the draft to nearly equal the depth of water, on account of the greater tonnage over which time costs will be distributed, may be more than nullified by the greater risks of striking sunken obstructions, or of being delayed by unexpected shoaling of the channel, by the difficulty of steering when the clearance is small and by the reduction of propeller efficiency."

Parenthetically, the writer, because of Colonel Shunk's mistaken assumptions, is compelled to disagree completely with the conclusions reached.

Constructor McEntee's experiments were made with models of different sizes, in channels of various cross-sections. In his "Report on Model Tests of Barges," he gives a number of curves of resistance for different ratios of length to beam of barges, for different ratios of boat and canal cross-sectional areas, and for different speeds. In these curves the ordinates are resistances in pounds per ton of displacement and the abscissas, speeds in terms of $\frac{V_m}{\sqrt{L}}$, in which, V_m represents the model speed, in miles per hour, and L,

the model length, in feet, "for use in extending model resistances to resistance of full-sized barges, for the reason that a large part of the total resistance follows the law that it is proportional to the displacement, provided the speeds of the model and the full-size barge are in proportion to the square roots of their length." Under this law, when the resistance of the water, in pounds per ton of displacement, is the same for the barge and the model:

$$V_m : v_m :: \sqrt{L} : \sqrt{l} \therefore V_m = \frac{v_m \sqrt{L}}{\sqrt{l}}$$

in which, V_m, v_m, L, and l are the speed and length of the barge and model, respectively.

Design of Cross-Section of Waterway.—To apply this formula practically let it be assumed that conditions require the use of a barge with a cargo capacity of 2 000 tons at a draft of 7 ft., and that a minimum speed of 5 miles per hour is necessary. What must be the cross-section of a canal which will permit this speed to be attained with a less resistance per ton of displacement than the 3-lb. draw-bar pull on an engine of a competing freight train?

Let it be assumed also that wharf lengths, canal curvature, lock lengths, etc., will permit the use of a barge similar to the Upper Mississippi River barges.* This barge at 7-ft. draft has a displacement of 2 575 tons with a load of 2 075 tons.† It is 300 ft. long, 48 ft. beam, and the ratio of barge length to model length is 21.43.

Under the required speed, $\frac{V_m}{\sqrt{L}} = 0.29$. From the relation, $\frac{D}{d} = \lambda^3$, in which, D and d = the displacements of the barge and model, respectively, and λ = the ratio of the lengths of the barge and model, $d = 523.3$ lb. This is slightly less than the displacement of Model 1984^A previously mentioned, at a draft of 6 in. For this value of $\frac{V_m}{\sqrt{L}}$, the model has a resistance of 2 lb. when the sectional area ratio is 7; of 2.6 lb. when the sectional area ratio is 5.6; and of 4 lb. when the sectional area ratio is 4.67.‡

The area of the wetted cross-section of the barge is 48 by 7 = 336 sq. ft. The canal of smallest cross-section to permit with certainty the speed to be developed with a resistance of less than 3 lb. per ton of displacement is evidently that for which the sectional area ratio is 5.6. The canal, therefore, must have a cross-section of 5.6 by 336 = 1 881.6 sq. ft.

Assuming that for local reasons the canal depth should be 3 ft. greater than the barge draft, or 10 ft., the required width becomes 188 ft., or 3.9 times the barge width.

In the discussions of the U. S. Tow Boat Board experiments, important elements which affect the channel widths and the depths required in practice (such as the effect of wind action, erosion of banks, etc.), are not considered.

* H. R. Doc. 108, 67th Cong., 1st Sess., p. 72.

† The form of the barge is quite similar to that of Model 1948^A, shown on Plate 3 o McEntee's "Report on Model Tests of Barges".

‡ "Report on Model Tests of Barges", Plate 22.

Squat of Boats.—In fixing the proposed channel depth, the "squat" or increase of draft of a boat when under way, must be taken into account, although it is slight at low speeds. Discussions on the squat of self-propelled boats* and of barges† have been published. For Hudson River traffic, using boats drawing 10 ft. of water and moving at a speed of 10½ miles per hour, the writer concluded that on account of the squat and of the impossibility of keeping the river bed completely free from obstructions, safe clearance cannot be assured with depths of less than 14 ft.

Bends in the Channel.—In rounding a bend, the channel width required for a single boat, or for a tow, is increased considerably beyond the beam of the tow. Herr Gröhe, Inspector of Waterways, Münster, Germany, reported to the VIth Navigation Congress, as follows: "We cannot go below a radius of 500 m. for canals, which allow of a rapid navigation with at most two boats in file." Several formulas have been developed to show this required increase in width. Observations on the movement of boats and tows in rounding bends were made in New York Harbor and in St. Mary's River. Two of the formulas were found to give results closely in accord with the observations, namely, that of M. Van Der Linden, $d = 4\left(r - \sqrt{r^2 - \frac{l^2}{4}}\right)$, in which, $d =$ the increase in width, in meters, necessary; $r =$ radius of curve, in meters; and $l =$ the length of the boat, in meters; and one deduced in the First New York District, River and Harbor Improvements, namely, $d = \sqrt{r^2 + l^2} - r$, in which, $d =$ increase required, in feet; $r =$ radius of curve, in feet; and $l =$ length of boat or tow, in feet.‡

Wind Action.—When boats are exposed to wind action from the side, they assume a more or less oblique position with respect to the axis of the channel, and thus require more channel width. In a published discussion on this subject§ the tractive and wind forces acting on the carrier are determined and the direction of the resultant is found. From the calculations, it was concluded that a self-propelled barge, 300 by 42 ft., with draft of 10 ft. and a free-board of 6 ft., moving at a rate of 4 miles per hour, and exposed broadside to a wind having a velocity of 9 miles per hour, will occupy 83 ft. of channel width; and that when, under similar conditions, the carrier is towed on a line, the tow-line will lie at an angle of about 10° 30′ with the axis of the channel.

Erosion of the Banks.—The effect of the wave produced by the movement of a boat through the water must also be considered, especially in restricted channels. The height of this wave is a function of the form and speed of the boat and of the dimensions of the waterway. The *Hendrik Hudson,* moving in the Hudson River near West Point, N. Y., at a rate of 22 miles per hour, produced a swell about 1 ft. high which became inappreciable at a distance of 1 000 ft.; passing Bogart Island near Albany, N. Y., in a channel, 770 ft.

* H. R. Doc. 1160, 63d Cong., 2d Sess., Inclosure G, p. 101.
† "Report on Model Tests of Barges", p. 18.
‡ See H. R. Doc. 391, 62d Cong., 2d Sess., p. 160.
§ H. R. Doc. 1387, 63d Cong., 3d Sess., p. 50.

wide and 12 ft. deep, the corresponding swell at the shore was 5½ ft. high, from hollow to crest.

When a boat is in motion there must be a constant movement of water, in a direction opposite to that of the boat movement, to fill the space the boat has just left. In restricted channels, even at low speeds, this water must move with a considerable velocity. In effect, the boat is climbing a grade; the water takes a slope down from bow to stern and, with the velocity due to the head created, rushes past the sides and under the bottom of the hull. The smaller the ratio between the sectional areas of the boat and the channel, the less will be the space through which this water must pass and the greater the velocity and consequently the head required to produce the velocity; and the greater the speed of the boat, the greater must be the velocity of flow. Bank erosion is caused by this flow in addition to the effect of the boat waves. If channel depths are to be maintained, this action must be considered and provision for it must be made.

Tows.—The methods used for towing cargo carriers has a marked influence on the type of towboats and carriers as well as on the required channel dimensions. In Europe, towboats are generally propellers and haul their tows by one or more lines. On the Rhine, a separate line from the towboat to each boat of the fleet is used, and each boat has its rudder and helmsman. This enables the boats to take a broad or a narrow formation, to suit channel conditions. In the eastern part of the United States, propeller towboats are used, hauling with a tow-line. In special cases, with one barge only, the towboat is fastened to the side of the barge; with two, between the barges. In the barge fleets of the Hudson River, bound to or from the Barge Canal, as many as sixty boats are made into a compact fleet, four or five abreast, and hauled by one or two tugs.

Boats that use the canal generally have full moulded forms with rudders. On the canal they are towed in small fleets of two to four. Boats that use the Champlain Canal and the crooked channel of the Narrows of Lake Champlain frequently are towed in a single line, and attached to each other by bridles which can be manipulated by winches, for steering the individual boats. On the Mississippi River, the barges are generally of scow form, without rudders, and are made into fleets by fastening the barges together as rigidly as possible with lines, thus forming a single floating unit which is pushed by the towboat. Stern-wheel steamers are preferred for this work, although tunnel-boat propellers are sometimes used. There is a record of a single tow, 258 ft. wide and 648 ft. long, made up of 27 coal boats, 8 coal barges, and 2 fuel boats, carrying about 25 000 long tons of coal. Such a tow is comparatively unmanageable and can be handled only down stream and in a broad channel. The U. S. Tow Boat Board found that the best formation for a tow of six barges was two abreast and three deep, the resistance per ton of displacement, in still water, with a speed of 5.25 miles per hour, for this formation being 3.13 lb., while in a formation three abreast and two deep the resistance was 3.54 lb. per ton. It is evident that to permit such a tow to operate channels must be broad and fairly straight.

Costs of Transportation.—These costs must vary with the conditions. If there is a line of water carriers running on a fixed schedule, with full cargoes offered in each direction, the cost of operation should be a minimum and a manufacturer located on the waterway can compare directly waterway and rail charges and advantages. If, however, the waterway is designed to serve manufacturers who must own and operate their own floating plants and, therefore, use the waterway intermittently, the economy of water carriage is decreased. For the towboats as well as for the barges, the cost in wages and in interest on the investment must be added for "lay" days and that cost plus the actual cost of operation must be apportioned to the total tonnage carried. Should the required run be longer than the working day, there will be extra charges for overtime, if not indeed for wages of a duplicate crew.

In studying a project for a waterway all the foregoing elements of cost must be given due weight. Conditions which control the cost of the construction and maintenance of channels in natural or artificial waterways differ so widely that no general equation of costs can be given, just as at present there is no reliable general formula for the resistance of boats. When the variable factors are known, however, deductions which are approximately correct can and should be made.

In comparing the economic transportation value of an existing or proposed waterway with that of a railway, the special conditions pertaining to each must be considered, bearing in mind that the addition of one or more trains per day on a railroad already in operation causes an insignificant increase in the general overhead operating costs. Data as to railways are widely scattered.*

C. McD. Townsend, Col., U. S. A. (*Retired*), M. Am. Soc. C. E., gives† an enlightening discussion on this subject and makes direct comparisons between rail and water transportation. In it he makes the statement:

"The increase in size of the freight car has enabled the locomotive to haul 200 tons of freight at the rate of 5 miles per hour with the same application of force that was required on the canal boat also carrying 200 tons and moving in the contracted waters of a canal at the rate of 2 miles per hour".

In this case, however, he compares efficient modern railway equipment with out-of-date waterway equipment. From the table compiled by Mr. Lavis,‡ it is found that the hauling capacity of a Consolidation engine (tractive power, 60 900 lb. to 62 500 lb.), under ordinary conditions, on a road with grades of 0.5%, moving at 20 miles per hour, is 2 160 tons, equivalent to a train of thirty 50-ton cars, carrying 1 500 tons of freight. Mr. Lavis also states:§

"For trains of loaded 50-ton cars, all of the same type, the resistance at ordinary freight train speeds is often as low as 3 lb. per ton, whereas trains of miscellaneous types and sizes, including some cars as light as 25 to 30 tons

* Much information is available in "Railway Estimates", by F. Lavis, M. Am. Soc. C. E., 1917; further information may be found in "The Maximum Weights of Slow Freight Trains", by C. S. Bissell, M. Am. Soc. C. E., and the discussion thereon, *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 303, and in "The Analysis of Cost of Freight Service, Grand Trunk Railway Company of Canada", by J. P. Newell, M. Am. Soc. C. E., and the discussion thereon, *Transactions*, Am. Soc. C. E. Vol. LXXXVI (1923), p. 351.

† "Rivers and Harbors", Chapter XV.

‡ "Railway Estimates", p. 445.

§ *Loc. cit.*, p. 444.

and some empties, will have a resistance as high as 6 to 9 lb. per ton, the usual average, however, being about 6 lb. per ton."

As stated previously, the resistance offered by a fleet of six ordinary Mississippi River barges, weighing 141 tons each, and with a capacity of 717 tons each at 6-ft. draft, moving in still water at a rate of 5½ miles per hour, was 3.13 lb. per ton of displacement. Two barges of improved type, moving at the same speed developed a resistance of only 1.87 lb. per ton; each of these barges weighs 500 tons, and at a draft of 9 ft. carries 3 000 tons.

A rough comparison between the freight-carrying capacity of a double-track railroad and that of an adequate sea-level canal may be made. The usual minimum headway allowed between trains is 30 min. If this headway is taken as 20 min., 72 trains per day can be run each way. If forty 50-ton cars are allowed per train, instead of thirty as previously mentioned, the carrying capacity per train will be 2 000 tons, and the total freight-carrying capacity per day for a single track will be 144 000 tons. A safe interval for boats moving at a rate of 5 miles per hour on an adequate waterway is 5 min. Under these conditions, 288 boats could arrive at a given terminal per day. If these boats are 3 000-ton carriers (the capacity of the type of barges used on the Upper Mississippi River at a draft of 9 ft.), this would make the one-way freight-carrying capacity of the canal 864 000 tons per day, or six times that of the railway.

The Future of Railway and Waterway Transportation.—The writer is convinced of the advisability of making a fuller use of the natural advantages possessed by the United States in its waterways, as a means of providing the economy in transportation so much needed for the nation's life. He is equally convinced, however, that the opening of inadequate waterways means only a waste of public funds. He hopes that this paper may serve to invite attention to facts which seem to have been overlooked by the radical enthusiasts among both the railway and waterway advocates, and that a further discussion of the subject may form a step toward that most desirable end—the substitution of co-operation for destructive competition between waterway and railway carriers.

DISCUSSION

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—In his opening paragraph the author presents the conclusion that a new waterway line of transportation should not be established unless the need for it can be shown and unless, "if established, it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs". There is another element which should sometimes be considered when the advisability of such a line of transportation is being studied, and by virtue of which many a project may be found advisable, despite the fact that it cannot, perhaps, for many years in the future, produce revenue to meet in full the interest charges against it. This element is the project's contribution to general prosperity as measured in terms of population, of business, and of property values. It becomes a factor because the expenditure of funds in its construction, and the utilization of the facilities which it provides when it is completed, bring increase of population; and to the extent that the growth in population of the zone coming under the influence of the improvement is thus accelerated, to the same extent will land values increase and wealth be created, which may fairly be weighed against the cost of the improvement when the advisability thereof is under consideration. Society, in other words, may reap material advantage from many an enterprise that would be condemned by the stockholder who weighs only the cost of the service against the prospective revenue.

It is, no doubt, this "benefit to Society" that has led to many large expenditures by the Government, without hope of financial return, for navigation and other projects which could not have been privately or locally financed; and it is this same consideration which in the past history of the country has prompted so many of the bonuses to railroad and other enterprises, by cities, counties, and States, as well as by the United States.

As illustrative of the foregoing proposition the following is taken from a paper entitled "History of the Conversion of the River Clyde into a Navigable Waterway, and of the Progress of Glasgow Harbour from Its Commencement to the Present Day",† by James Deas, Engineer of Clyde Navigation, read at the International Engineering Congress of the Columbian Exposition, 1893:

> "These improvements have raised Glasgow within the short period of 90 years from a second-rate inland provincial town, with a population of 77 000, to be the second city of the Empire, with a population of 658 073, and the chief seaport of the west of Scotland; increased the value of lands on the river's sides from Glasgow seaward a hundred-fold; created the burghs of Govan and Patrick; given wealth to thousands, and the means of life to hundreds of thousands of the inhabitants of the northern portion of Great Britain, and emphasized in a marked degree the local epigram, 'Glasgow made the Clyde and the Clyde made Glasgow.'"

In the same vein, the late W. Henry Hunter, M. Am. Soc. C. E., Chief Engineer to the Manchester Ship Canal, in a paper entitled "Artificial Water-

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

† *Transactions*, Am. Soc. C. E., Vol. XXIX (1893), p. 129.

ways in Great Britain",* presented at the International Engineering Congress, held at St. Louis, Mo., in 1904, said:

"Apart from some such State aid, it is difficult to see how the scheme can be financed satisfactorily, particularly after the experience of the working of the Manchester Ship Canal, of which it may be truly asserted that such working has been and is the occasion of almost incalculable benefit, both direct and indirect, to all within the sphere of influence of the waterway, except the shareholders who provided the money for the inception of the project and the construction of the greater part of the undertaking." * * *

"* * * The operation of the Manchester Ship Canal has directly or indirectly affected beneficially the whole of the greatest industrial district on the face of the earth; a district of which the present population does not fall far short of 10 000 000 persons; while in the more limited area of which the City of Manchester is the heart and center, many industries have been saved from extinction and many others from decline and ultimate decay". * * *

"Now, not only has decline been arrested, and decay averted, but lines of new and vigorous growth have shot and are shooting out on every side, north, south, east, west, there is no exception; expansion is universal; old industries have been revived and new industries introduced; the change is definite and impressive, and has been due solely to the effect and influence of the canal."

C. H. Markham,† Esq.—It is doubtful whether there is any important public question about which so little reliable information is given out and which is so little understood by the public generally as the question of the present use and prospective development of inland waterways. When efforts are being made to obtain Congressional appropriations for inland waterway improvements, it is usually not supporting information that is wanted by their advocates, but appropriations. Of course, before these projects are acted on by Congress, the reports of the United States Army Engineers are received, but these reports are usually not given such consideration as will interfere with obtaining the desired appropriations.

That is a strong statement, but it can be illustrated by the case of the proposed Illinois State Waterway Improvement between the Chicago Drainage Canal and the Illinois River, which is part of the plan to provide a navigable channel between Chicago and a connection with the Mississippi River at Grafton, Ill., and thereby make barge transportation possible between Chicago and New Orleans, La., a distance by water of 1 610 miles. As far as the real promoters of this project are concerned, the relation between the saving in the total cost of transportation that can be expected and the cost to the public of the proposed waterway has not been given the slightest consideration.

There is no valid objection to the development and use of a waterway where the tonnage that can be expected to move over it on its completion is sufficient to justify the expenditure of public funds raised by taxation and, therefore, paid by all the people. That sort of improvement is in the same category with the improvement of ports and harbors by the Federal Government and the contributions which the Federal and State Governments make to the construction and maintenance of public highways. However, it would hardly seem possible that those responsible for the making of Federal and State expenditures for highway improvements would contribute large sums

* *Transactions*, Am. Soc. C. E., Vol. LIV (F) (1905), p. 190.

† Pres., Illinois Central R. R., Chicago, Ill.

of money for the construction of a highway located for miles through a section of the country where there would be little or no travel. Such contributions are made only under conditions where there is a reasonable prospect that intensive use will be made of the highway when it is completed. The same study should be made of proposed waterway improvements and should govern the appropriation of public funds for such improvements.

The advocates of the development of inland waterways almost invariably seek to reinforce their arguments with the statement that the railroads have broken down and shown themselves unable to handle the traffic offered from time to time. If it is pointed out that during the last two years (1923-24) the railroads have been furnishing all the increased transportation required, they reply that the future rapid growth of the business of the country will find the railroads unable to meet future requirements, and hence it is necessary to supplement the nation's domestic transportation machinery with inland waterways. Still another argument is that the development of inland waterways will relieve the railroads of a large part of their freight traffic that is carried at low rates, such as coal, grain, lumber, and the like, which, they assert, is unprofitable to the railroads.

The truth about the latter argument is that almost any American railroad would starve to death if its heavy freight business were taken away from it, because that is the freight that pays best. A railroad would rather handle coal at one-fourth the rate per ton per mile that it receives for handling merchandise. Coal moves in carload shipments, and cars are loaded to the maximum, thereby reducing the proportion of dead weight that has to be carried; it is handled in heavy train loads, making for economy in train operation; and there are comparatively few claims to pay. Altogether, even at comparatively low rates, it is the most profitable business the railroads handle. That ought to dispose of the argument that the development of waterways is desirable from the standpoint of relieving the railroads of that part of their business on which comparatively low rates are charged.

Concerning the argument that waterway development is needed to help meet the transportation requirements of the future, there is this to be said: It is beyond physical possibility that inland waterways can ever be developed to the extent of making any considerable contribution to the total volume of freight traffic that must pass between points in the interior of the country. The total volume of both foreign and domestic commerce handled into and out of all the ports of the United States and its dependent territories—including export, import, and coastwise business, but eliminating the Great Lakes and not counting coastwise traffic twice, as is done in the keeping of records—was about 96 000 000 tons in 1923. As against this, the railroads handled 1 277 000 000 tons. The Illinois Central System, comparatively a small railroad, alone handled 65 000 000 tons, or about two-thirds as many tons as the total number of tons of import, export, and coastwise freight handled into and out of all the ports of the country, except the Great Lakes. That ought to dispose of the argument that inland waterways—a relatively small factor in the nation's total water-borne commerce, which is principally foreign commerce—can ever be of material assistance in helping the railroads to meet

the increased transportation requirements of the future. The main reliance for domestic transportation must still be placed on the railroads, as it has been in the past.

Although it is true that the railroads for some time have been handling about all the traffic possible with their present facilities, there is no conceivable limit on the amount that can be handled if they are enabled through adequate earnings to maintain their credit and increase their facilities. Their business has increased 800% in the last forty years, and they have met that increase creditably. That is assurance that they can do as well in the future if given the support which the public interest demands.

As regards the savings that can be effected by the use of waterways as compared with the costs of providing and maintaining them (which is the real test of waterway economy), the speaker submits the following:

The total capital cost of the New York State Barge Canals, up to June 30, 1924, including the Erie Canal and the various branches, was $168 053 962.31. Capitalized at 4%, that makes an annual interest charge of $6 722 158.49. The expense of operating and maintaining them in the fiscal year ending June 30, 1924, was $4 099 975.67. This, added to the interest charge on capital investment, makes an annual charge against the traffic carried by the New York State Barge Canals of $10 822 134.16. The entire New York State Barge Canal System in the calendar year 1923 carried 2 572 635 tons of freight, so that the annual charge for interest on investment and the cost of maintenance and operation averaged $4.21 per ton. Now, a charge is high or low by comparison. The average revenue received by the railroads in 1923 for each ton carried was $1.97, and this was in full payment for all the service, not merely the cost of providing the roadbed over which their trains could be operated. The charge against the traffic of the New York State Barge Canals of $4.21 per ton covers only the cost of providing and maintaining the canals and does not include the boat charges for handling the freight.

The experience on the Monongahela River is often referred to. In 1923, the Monongahela River carried 23 562 925 tons of freight, of which 18 709 084 tons was coal. Its traffic in 1923 was larger than ever before, an increase of nearly two-thirds over 1922. All that business is short haul. It is carried in vessels of the owners of either the coal mines or the steel companies to which the coal is brought and from which small quantities of steel products are carried away. The United States District Engineer in charge of the Monongahela River improvement states that there are no common carriers operated on that river. To all intents and purposes, the Monongahela River improvement is a plant facility for the large corporations that use it. The cost of operating and maintaining the Monongahela River improvement project in the fiscal year ending June 30, 1924, was $819 273, or about 5.7 cents per ton on the 1922 traffic of 14 407 129 tons, or 3.5 cents per ton on the 1923 traffic of 23 562 925 tons. This is simply the cost of operating and maintaining the improvement, not counting an interest charge on the expenditure of nearly $10 000 000 for canalizing the river, providing the regulating works, etc., nor the cost of operating boats.

It has been suggested that one reason why a greater amount of traffic has not been developed on the Ohio River is that the work has not been completed as quickly as was contemplated when the first appropriations were made.

The present project for the improvement of the Ohio River was adopted in 1910. At that time it was estimated that it would cost $63 731 488 to complete the series of fifty-two locks from the head of the river at Pittsburgh, Pa., to its junction with the Mississippi River at Cairo, Ill., a distance of 968.5 miles. According to the latest report, thirty-seven of those locks have been completed, the United States Army Engineers are working on eight others, and seven are untouched. The Federal Government had spent, to June 30, 1924, on all projects, a total of $76 788 387. It is estimated that it will cost about $43 000 000 to complete the work. If that is true, the entire project will have cost, in round figures, $120 000 000, or virtually twice the original estimate.

There is not enough water in the Ohio River now to navigate those parts that have not been provided with locks, so that for a considerable length of time the river will be out of use, until the volume of water increases. The Ohio River handled in 1923 only 8 280 520 tons of freight. How much more it will handle after the project has been completed is a matter of conjecture.

Advocates of the development of inland waterways often refer to the use that the European countries are making of their waterways, on one hand, and the use that is being made of the Great Lakes in the United States, on the other. Whenever the navigation of rivers in European countries is referred to, it is the German Rhine that is meant, as the Rhine is the only river in Europe that is being used extensively, with the exception of the Elbe in Germany. In this connection, it is highly important to note that Germany was a thickly populated country with good sized centers of population long before railroads were known. All the towns and cities and commercial interests that had been developed up to the time of the railroads were located on the waterways. In addition, the Rhine flows through the important Westphalian coal fields, and the location of those coal fields determines the location of industries along the river in the Ruhr District. The iron ore for use in the steel plants of the Ruhr comes by sea to Rotterdam, is delivered to the German Rhine at Emmerich, and thence moves up the Rhine to the German industries.

Starting with the double advantage of the location of not a few considerably sized towns and cities on the Rhine and the natural location of the river with reference to the large coal fields and dependent industries, when the time came for an intensive development of the railway systems, what was done? The railroads of Germany, like the railroads of America, never began to assume development proportions worthy of the name until after the Franco-German War. That experience, perhaps, with inherited traditions going back many hundreds of years, pointed to the direction in which railway construction should take place, and that direction, according to the German mind, was toward the Belgian and French frontiers. As transportation agencies the use of railroads was subordinated to the use of waterways and to the fact that when needed they might be available for military purposes. To that

end not only were the railroads constructed toward the frontiers, but the Government owning the railroads made the rates high enough to encourage the use of the waterways and discourage the use of the railroads. How can a comparison that is worth anything be made between the use of a waterway developed under those conditions and the theoretical use of a waterway under entirely different conditions in the United States?

Concerning the reference that is often made to the low cost of freight transportation per ton-mile on the Great Lakes, it is hardly fair to compare the transportation of iron ore, coal, and other bulky commodities in 12 000-ton vessels, using the most modern labor-saving devices for loading and unloading the tonnage in huge volume, with the transportation of hats and caps and boots and shoes and all the various other articles of general commerce by the railroads, across rivers and plains and over mountain tops. Yet that is the kind of argument that often is broadcast by advocates of inland waterway development.

Take another inland waterway, the Lower Mississippi River, which is virtually paralleled by the lines of the Illinois Central System. During the World War, the Director-General of Railroads recommended an expenditure of about $12 000 000 to provide transportation on the Lower Mississippi River to take care of the business that the railroads at that particular time were unable to handle. The appropriation was finally made, and the Federal Barge Line has thus been in use since 1919. It has been properly regarded as an experiment to determine the possibilities of barge operation on the Mississippi River.

The Illinois Central System from the beginning has co-operated with the management of the Barge Line in every conceivable manner—establishing interchange facilities at meeting points and making agreements concerning the division of through rates which have been entirely satisfactory to the Barge Line management.

An old story will perhaps illustrate some of the reasons for this policy of the Illinois Central System. A Western Congressman, in the early days of Populism, talking with a friend in Washington, said that he had just received news about the alarming growth of Populism in his State, and that he must go back and look into the matter. His friend said, "I suppose if you go back and find they are that strong, you are going to fight them." "No," he said, "if I find them that strong, I am going to join them."

From the beginning the speaker has had no faith in the success of the Mississippi experiment except on a basis of continued subsidization by the Government or the creation of conditions which in the end result in the total cost of transportation being greater than if it is carried by rail. Not having counted on the success of the experiment, he did not want it to be said, when it did fail, that the Illinois Central System had made any contribution toward that failure. On the other hand, if by any chance it did succeed, then the Illinois Central System would be in a position where, if opportunity offered, it might share in some of the credit.

As a member of the Waterways Committee of the United States Chamber of Commerce which made a report several months ago on the question of the

development of waterways and their co-ordination with the railroads for transportation service, the speaker joined in its recommendation that, to determine more fully the possibilities of inland waterway transportation, the Secretary of War should be given the necessary authority and funds to operate the barge lines on the Mississippi and Warrior Rivers in accordance with good commercial practice. Shortly afterward Congress appropriated several million dollars for use under the direction of the Secretary of War in handling the business of the Mississippi-Warrior Service along commercial lines. It was well argued by Brig.-Gen. T. Q. Ashburn, U. S. A., who is in charge of that work under the Secretary of War, that one of the reasons they were unable to handle the business successfully was that if in one year they made a profit, it went into the general fund, and they were not able to use it the next year, if there was a loss. Another argument was that under existing conditions they were not in a position to act freely because of the necessity of turning back money to the United States or going to Congress for various other needs.

From the beginning of barge line operation to June 30, 1924, the Mississippi River line took in $11 532 250 and paid out $14 541 496, or $3 009 246 more than was received. In addition, depreciation charges amounted to $1 791 160, making a net deficit, including depreciation, of $4 800 406. Something ought to be allowed for pioneer waterway development, as General Black has said, but the fact is that in the fifth year of operation of the Barge Line—after the completion of its pioneer work—there was a deficit of $254 000 before depreciation and of $679 000 after depreciation; and nothing is included therein to represent the cost of providing and maintaining a navigable channel in the Mississippi River, which has been no inconsiderable sum.

At present, the Barge Line is having great difficulty in getting its barges over the sand-bars on account of low water. There is no navigation between St. Louis, Mo., and Cairo, and the movement of tonnage between Cairo and New Orleans has been very light and very slow on account of the lack of water in the river. It does not do to say, however, after using a vast amount of the nation's money, that the Barge Line's inability to earn a return is because the river is too high, or too low, or has ice in it. Those are hazards that the business ought to be expected to encounter; it is one of the things that ought to be taken into consideration when the Federal Government undertakes to spend public money in developing the waterways.

In this same connection reference has been made to the fact that in the early days of the railroads they were built largely by Government subsidies. As for the money that was advanced by the Government to the transcontinental lines, every dollar of it has been repaid with interest. It is true that the Government gave some of the railroads land in addition to money, and, of course, the land has been kept. The Illinois Central was one of the land grant roads; it was begun in 1851. Its promoters agreed to build it if the State would secure from the Federal Government a grant of land for a right of way and a certain number of alternate sections on either side. The Federal Government ceded to the State of Illinois more than 2 500 000 acres of land to secure the construction of a railroad in Illinois from the Great Lakes to the Ohio River at Cairo, and also ceded to the States of Tennessee, Missis-

sippi, and Alabama certain grants of public lands to secure the construction of a railroad from the Ohio River to the Gulf of Mexico, at Mobile, Ala.

The Illinois Central accepted the grant from the State and took the 2 500 000 acres of land. The Government required, however, that the railroad should forever carry its property and its passengers at certain reductions in rates. The first requirement was that the railroad should provide the roadway and the Federal Government should be allowed to operate its own cars and engines over it. Somebody had a theory that that could be done. Afterward that part of the agreement was changed, and it was put on the basis of reduced rates. Since that arrangement has been in effect, the Illinois Central has given the Government about $7 000 000 in rate reductions. At the time there was no demand for land in Illinois because there was no transportation. As soon as the Illinois Central accepted these 2 500 000 acres, lying in alternate sections, the Government advanced the price of the remaining sections to $2.50 per acre, and, when the land began to sell, the price was advanced by degrees above that, so that although the Government may have given the land to the Illinois Central, it was a good thing for the Government, which repaid itself by the stroke of a pen, not to speak of the $7 000 000 returned in reduced rates.

As to the State, the Illinois Central accepted an agreement by which it was to pay 7% of the gross receipts of its charter lines into the State Treasury. The result is that the Illinois Central System has already paid the State $60 000 000. Last year (1923) alone it paid more than $3 500 000 in addition to some other taxes.

It cannot be said that the Government through its land grants and other subsidies did so much for the railroads as might be thought even if merely the fact of having made these early contributions is taken into account, without considering that every dollar of that money has been repaid, in some cases many times over.

Concerning Colonel Slattery's reference to rates between water route points and non-water route points, it should be remembered that it was not until the railroads were built that there was any considerable transportation except by water. The railroads were built under the same conditions that highways are being built to-day. A great deal of money is now being spent in Illinois for the improvement of public roads. They are being built alongside the railroads, because that is where the towns are and that is where the business is. When they come to the towns that are already served by the railroads, no criticism is made; it is a natural development—one of the things that is occurring in this age—and one would be foolish to protest against it.

That is exactly what happened in the construction of many railroads. Take, for example, the construction of the railroad from Memphis, Tenn., on the Mississippi River, down through the delta of the Yazoo. When this railroad was started the only navigation possible was at the high places; the river, as it rose, inundated the whole country. The railroad, therefore, was built to those points where there was some business, and the only way it

could live was to take the business there. When business slackened boats could move to other fields, but the railroad had to get enough business to live where it had laid down its rails; and to get the business the railroad had to make rates as low as the boat rates. That was in the beginning. When it came to making rates between points in the interior, the railroads naturally made the rates on the basis of what the traffic would bear.

A wise traffic man knows that it is his business to encourage the movement of traffic along artificial lines. In the present rates through the Panama Canal there is seen a correction of the difficulty Colonel Slattery has referred to, because under the present law a railroad having once reduced its rates to meet water competition cannot raise them again without the consent of the Interstate Commerce Commission. It is not permitted to raise them because the use of the water rate has been discontinued.

If the railroads were to raise their rates to the point where the business would be turned over to the highways, there would not be any competition; the railroads would go out of business.

Further, there is no such thing as real competition between a waterway and a railroad. There is no competition between the railroads and the boat lines on the Great Lakes for the ore business. Every pound goes by water, and it cannot move by rail unless there is some particular object that cannot be got into the hold of a ship. Whenever they are left without interference, either one or the other will eventually take the business. There are never two waterways side by side and competing as two railroads would or two highways.

In summary: It is not humanly possible at any figure of cost within the reach of one's imagination to develop inland waterways within the next twenty, thirty, forty, or fifty years to an extent that will make the slightest impression on the total amount of tonnage that is now being handled or that the railroads will be required to handle in the future.

There is no limit to the amount of tonnage that the railroads of the United States can put themselves in position to handle. It simply means more tracks, more cars, more engines, larger terminals. That can be done. Waterways cannot be built across mountains; they cannot be put where there is no water. Railroads can be built anywhere, and they can go on and on, and on, and furnish all the transportation that the country needs, when it needs it. All that is necessary is to treat the railroads in the matter of regulation so that when the end of a year comes there will always be enough money left, after paying operating expenses and fixed charges, to encourage continuous investment in their securities.

EDWARD BURR,* M. AM. SOC. C. E.—This paper is of value in drawing the attention of the professional public to the subject of waterways, which is always present, but usually slumbering; it notes those technical requirements that are essential to the economic planning and operation of waterways or any other transportation system, and refers in detail to the necessity for the consideration of the economics of water transportation. Those economics must

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be taken into account in a broad way as well as in detail in determining the advisability of undertaking any waterways improvement or extension.

The controlling element in the economic consideration of these matters is the public ownership of these highways—of water transportation routes. There is no question as to the desirability of public ownership of some publicly used facilities, nor of retaining in private ownership other public utilities. The line between the two classes is indeterminate and shifting; but the ownership of certain transportation routes on water and on land has always been vested in the public, and the wisdom of this broad policy cannot be questioned at this time.

The earliest land transportation routes were mere trails, free to all. Whatever cost or expense of labor was involved in opening and maintaining them was borne by the public. In addition to the public highways, there have been some privately owned land highways—toll roads. They were not striking financial successes; most of them have long since been absorbed by the public. The use of public-owned highways has been free to the public, and the charges are a general assessment on the public, more or less broadly spread, depending on the scope of the benefits which they render.

There is a wide analogy between highways on the land and those on the water. In the same early periods, the water routes of the earliest settlements along the coast were of even more importance than those on land, especially for the transportation of cargo. Similarly, in the early period, the improvement by private capital of some routes and the construction of others where none existed, were not only permitted but encouraged by the public. These privately owned waterways have followed much the same course toward public ownership as did the toll roads on the land, until, at present, there are few that are thus owned and operated. The reasons in the case of water routes are much the same as in the case of land routes.

The full and free use of natural waters for navigation and other purposes has long been recognized and carefully protected as the country developed. These water routes were improved and extended by public authority, and their use has continued free to the public. This policy has come down through 100 years of Federal control and, no doubt, will continue into the future. It has had much to do with the economics of the development of water routes at the public expense.

As routes were improved or extended, such studies as were practicable as to the economic benefits to result from the expenditures of public funds have been made. Those who have had experience in making such studies are fully aware of the difficulties of arriving at a satisfactory solution—one of such reliability as fairly to forecast the future, not only as to the commerce to be developed but as to the economic benefits, special and general, to accrue to the public, the owner of the routes. That this difficulty was not peculiar to the public officers who made such analytical studies, is clear from the records of those land highways and water highways which in the past have been improved at the expense of private capital. It is evidenced by the disappearance of nearly all those routes, and by the small financial return on those which still continue in existence and operation. The explanation rests on the fact

that, to-day, and in the recent past, few waterways have made an adequate direct financial return for the private capital or public funds expended on them and that the indirect benefits, especially from public, free waterways, are impossible of evaluation.

The question arises as to why these improvements should be undertaken, when no direct benefit to the owner (the public) can be shown; and when in many cases little indirect benefit to those who might need them is in evidence through their general or systematic use.

The same question may be asked in regard to public highways on land. In how many cases of land highways, of public roads, can an economic analysis be made which will justify the construction through the showing of any adequate return direct to the owner of the roads, the public, or to the public treasury, or even any adequate indirect return?

What is it that leads the public to insist on having these facilities, at the public expense, when no analysis can prove any adequate financial return, direct or indirect? The answer is found partly in the public desire to control certain necessary public facilities rather than to allow them to pass under the control of private capital. That, however, is not all. Every one through familiarity with the results flowing from the extension and betterment of public land transportation routes, recognizes the material indirect benefits to the public and to the public funds following all worthy improvements in transportation facilities, although such benefits may not be susceptible of reliable measurement by the financial yard-stick. Increased commerce, trade, and property values and decreased costs may be difficult to predict or even to apportion afterward, but they are not the less influential in directing action. The vision of the men who built the Erie Canal was clearly verified by its benefits to the City and State of New York and to the Nation. It was not measurable by the financial return or by the amount of traffic. The variation of the summer and winter rates between Buffalo and New York is not forgotten to-day.

Some years ago, a locomotive was needed for construction work on the Columbia River, fifty miles east of Portland, Ore. It was ordered by wire and assiduously traced for prompt forwarding but, disregarding urgent requests for delivery, the railroad company carried it past its destination to Portland, whence it was returned three days later. This minor incident merely illustrates some features of the rail rate situation and their effects. The rail freight rate was the through rate to Portland, a water competitive point, plus the local rate back to the destination, and the railroad company felt impelled to justify the rate by actually making the haul. The entire question of transcontinental rates (to water competitive points) *versus* rates to intermediate points has since become well known through the Inter-Mountain Rates cases.

Water routes have not been developed or improved for the deliberate purpose of regulating rail rates, but they have had and still have the effect of bringing about a reduction of such rates to competitive points. Much of this benefit is almost ancient history, but even now the railroad companies are seeking a

reduction in transcontinental rates to meet the competition of the improved water route through the Panama Canal.

With equality in rates for the line haul and with greater convenience in pick-up and delivery by the railroads through the admitted greater flexibility of rail terminals, it is to be expected that they will secure by far the greater part of the business at some competitive points. The waterways, however, have justified the public's confidence and willingness to spend its money on them, although they may carry little actual commerce. To use a naval phrase, they have the value of a "fleet in being". Private operations over them may not be spectacular in tonnage or in financial returns, and Government operations, perhaps, may be expected to be even less so, but to illustrate general results by a specific case, the work on the Columbia River already mentioned has returned to the public many times the sums spent on it, although its traffic record is inconsequential compared to that of the parallel rail routes.

The speaker is far from advocating the reckless or extravagant expenditure of public funds on public works of any description. He does feel, however, that the improvement of water routes must not rest alone on the expectation of adequate direct financial returns to the owners of the utility—the public. Unimpeachable justification of the cost of such improvements by any possible economic analysis is not to be expected in the general case. Even judgment founded on long experience, observation, and study is not infallible. What is needed is some portion of that vision or second sight which is so uncommon among men of all classes, including engineers.

William G. Atwood,* M. Am. Soc. C. E.—This paper performs two distinct and valuable services for transportation engineers. It collects and summarizes much scattered information regarding the design of waterways and waterway equipment, but, what is more important, it calls attention to some of the principles which should govern a decision as to whether a given transportation facility should be constructed and if so as to what form this construction should take.

Generally, transportation engineers have been and are specialists in railway, highway, waterway, or, more recently, air transportation, and many of them have been violent advocates of the type with which they are best acquainted. The development of the art of transportation makes it necessary that the engineer divest himself of any bias that he may have in favor of a single method of giving this service, that he attack the problem with an open mind and develop a co-ordinated system in which all methods may have a part. The services of specialists are still necessary after the general plan is adopted.

Custom and the basic characteristics of the different media have divided the transportation of the United States into two general classes; (1) Waterways and highways, generally constructed and maintained by some governmental agency, on which the traffic is moved by privately owned and operated equipment; and (2) railways where all the financing, construction, maintenance, and operation are privately performed and controlled, under Gov-

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ernment regulation. The necessity for co-ordination in the design and operation of these two classes of facilities is only beginning to be generally appreciated.

Exceptions to these general classifications exist, such as the operation of an ocean fleet, of the Mississippi-Warrior River Barge Line, or the construction and operation of the Alaska Railroad by the Federal Government. Subsidies were also granted by the Government to the early transcontinental railways, by certain of the States to such roads as the Illinois Central and the Nashville, Chattanooga, and St. Louis. The Cincinnati Southern was constructed and is still owned by the City of Cincinnati, and many subsidies smaller in amount were given to the early railways by local governments and individuals, but the operation remained in private hands.

Conditions in other countries are quite different. In many of them not only are the waterways and highways built by the Governments, but the waterways are frequently, partly at least, Government operated and in many cases the railways are both Government owned and operated. Under these conditions the co-ordination between the different methods of transportation is generally much better than in the United States.

General Black's rule that a transportation facility should not be constructed unless it will "produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs", seems to place an unwise restriction on construction. If a transportation facility will produce these results and promise a fair profit to its promoters, it will be constructed by private capital; but the development of a country may require the opening of transportation routes which will not fulfill these requirements.

At the time many of the early American railways and canals were built, they could not earn operating expenses, but the general good certainly justified their construction from a purely economic standpoint to say nothing of the political and cultural benefit which resulted. The development of the River Clyde and the construction of the Manchester Ship Canal and of the Panama Canal are other examples of expenditures fully justified by the general benefit which resulted, but which would not probably qualify under General Black's rule. In Europe the needs of the State, or what were thought to be such, also had their influence not only on the decision whether or not to construct, but on the selection of the type of facility and its design. For example, the railway system of the old Austro-Hungarian Empire was designed and built to centralize traffic in Vienna and Budapest for strategic reasons and in order to aggrandize those cities, although railways on more direct routes would have much better served the traffic needs of certain parts of the Empire.

In the case of privately owned facilities the user pays all the transportation costs, but with publicly owned facilities a part is paid by the user and the remainder by taxation. Eventually, these costs are paid in the cost of living by the general public and the construction seems to be justified if the sum of the tangible and intangible benefits equals or exceeds the cost. The construction or operation by governmental agencies of facilities which will qualify

under General Black's rule will compete with private initiative and, therefore, are contrary to the principles on which the whole economic structure of the United States is built. Such methods are justified only when these requirements cannot be met and when it appears that intangible benefits of sufficient value will result.

After it has been decided to construct a new transportation facility, the selection of the type should be approached with more care and study than has generally been the case. The advocates of the various types frequently omit pertinent items from their estimates and forget that the cost of moving freight begins in the warehouse of the consignor and ends in that of the consignee. The fact is also frequently overlooked that interest on the construction cost and the cost of maintenance of the waterway and highway (all of which are paid by taxation) are just as much a part of the cost of transportation as the same items included in the railway rates. Too frequently these comparisons are made between the costs of moving a ton of freight after it is placed in the vehicle of the carrier. It is unfair to state that waterway transportation is cheaper for a given 100 miles when the actual cost of moving the freight is 0.3 cent per ton-mile by waterway, 0.9 cent by rail, and 4.0 cents by highway, if the cost of delivery to and from the waterway and railway terminals and the terminal handling charges are, respectively, $3.70, $3.10, and nothing. High terminal charges have had a great influence on the decline in waterway traffic; conversely, the elimination of terminal charges in highway transportation is one of the reasons for its success.

Much of the argument in favor of waterways in the United States is based on European conditions without consideration of the differences in the history of the development of transportation on the two continents. In Europe, the first transportation was by waterway and highway. The industrial districts were developed so as to reduce terminal costs to a minimum, and these districts were built up solidly. When the railway arrived, it had difficulty in furnishing terminal facilities equally convenient for the industries. In the United States the industrial districts were built around the railways and now waterways are faced with terminal difficulties similar to those with which the European railways were confronted. These difficulties may and must be overcome in order to secure the most economical and efficient transportation system. Two examples may be cited.

The City of St. Louis originally depended on the Mississippi and Missouri Rivers for transportation, but its general industrial development followed and depended on rail transportation. To return a part of the traffic to the rivers has required the development of more economical vehicles, the construction of expensive river terminals, and the construction of interchange facilities between rail and water. Even then, many industries do not find water transportation economical, in spite of 20% lower rates, on account of the high cost of delivering freight to the water carriers.

In the days of Richelieu, La Rochelle, France, depended on water and highway transportation; the city walls enclosed the port. When the railways

came, their terminals had to be located outside the walls and had no physical connection with the port. In order to "bring the wheels and keels together", it was necessary to construct an entirely new rail and water terminal and an industrial district 3½ miles away at La Pallice.

In the future, transportation engineers must see their problems on a larger scale than has frequently been the case in the past; they must always have in mind the use of any or all the available methods of transportation and must not overlook the fact that co-ordination and co-operation of the various methods are necessary in order to give the best service for the lowest cost.

J. R. Slattery,* M. Am. Soc. C. E.—Underlying this demand for the improvement of internal waterways—and it is a real demand—lies a condition exemplified by a few rail rates which will be quoted. These rates are not from the latest rate tables and, therefore, may differ somewhat from actual present rates. From Cincinnati, Ohio, to Evansville, Ind., on the Ohio River the distance is 270 miles; from Cincinnati to Gallatin, Tenn., which is inland, the distance is 273 miles. The rates to Gallatin are from 28.4 cents to 36.0 cents higher per 100 lb. than those to Evansville. From Pittsburgh, Pa., to Cairo, Ill., at the junction of the Ohio and Mississippi Rivers, the distance is 693 miles and from Pittsburgh to Dalton, Ga., an inland point, the distance is 689 miles. The class rates per 100 lb. from Pittsburgh to Dalton are from 31.5 cents to 63.5 cents higher than to Cairo. Innumerable other examples of such differences in rates between river points and inland points the same distance apart could be quoted.

In urgent demands that come before the U. S. Board of Engineers for the Improvement of Waterways, the claim is almost invariably made that such improvement would result in material savings in freight cost; possibly underlying this demand is the hope that the improvement will bring about a reduction in rail rates between points on the waterway, which very often happens after its completion. It is possible that in some cases the advocates of improvements are more interested in securing a reduction in rail freight rates than in actually securing service by boat.

There is some reason to think that if rail rates were based primarily on the length of haul with, of course, proper allowances for terminal charges, the conditions of the waterways would be very different from what they are. There are many who think, not without reason, that the establishment of uniform rates for certain lengths of haul regardless of whether the points between which the haul is made are water points or inland points, would result in a vast fleet of boats on American waterways.

Mr. Markham has brought out clearly the small use that is being made of some improved channels and the losses that have been incurred in attempting to operate boat lines on them. It would be interesting to determine how great the return on the investments in waterways would be, if to their account were credited the lower freight rates allowed by railroads on hauls between

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points along the stream. A vast trade moves by rail along many of these waterways between water points and on such freight a vast sum of money is saved through the lower rates allowed.

The Engineer Department of the Army has never favored using a waterway merely to pound down freight rates, but there is a strong sentiment urging that very thing.

C. I. Grimm,* Assoc. M. Am. Soc. C. E. (by letter).—In speaking either of waterway or of railroad transportation in general terms, a wide range is included and the author has covered fully the general aspects of the subject. A specific comparison of transportation on a particular kind of waterway with transportation of the same commodities by rail is the problem that generally has to be considered by those who are studying the advisability either of improving waterways or of using waterways already improved. The main purpose of this discussion is to present such a comparison in a field that is now of sufficient interest to warrant considerable attention.

The author recognizes that interest charges on the cost of an improved waterway as well as the expense of maintenance and operation, even though paid from the National or State treasury, are a proper charge against the cost of transportation—a premise which should be generally accepted. If a large tonnage is carried, these charges are relatively small for waterways such as the Great Lakes and for non-canalized navigable rivers such as the Lower Mississippi, but quite appreciable for canalized rivers; whereas for canals with numerous locks they may easily exceed the carrying cost of all traffic on the waterway until the time when this traffic approaches the limits of capacity.

Specific comparisons of rail and water costs for the coal and ore movement on the Great Lakes are hardly necessary to demonstrate the economy of lake transportation; on the other hand, it is generally true that purely artificial canals with numerous locks do not compare favorably with railroads, and that their construction is advisable, if at all, only where they form a short connecting link between waterways that afford more favorable conditions for transportation and that have a large existing or prospective traffic in bulk freight. In view of the foregoing and of the fact that at present the navigable inland waterways of the United States consist largely of canalized and non-canalized rivers permitting drafts of 6 to 9 ft., comparisons between transportation on such waterways and on railroads are of particular value; and as the principal tonnage on these waterways is coal, this discussion will be limited to consideration of the transportation of coal on a canalized river, as compared with railroad transportation. It is understood that what applies to coal applies to any similar commodity that might move in equivalent quantity and in carriers of the same type.

The method of transporting coal by river is almost exclusively in groups of barges with capacities of 500 to 1 000 tons, pushed by towboats. The larger sized barges are more economical to operate where large tonnages are handled and a draft of 8 ft. or more is available.

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The dead weight of steel barges with capacities of 500 to 1 000 tons is approximately 20% of their live load capacity, or 400 lb. per ton of capacity. For steel coal cars this percentage is about 40%, equivalent to 800 lb. per ton of capacity. The cars will cost somewhat more per unit of dead weight than the barges. At present prices barges will cost not more than $20 per ton of capacity, whereas coal cars will cost at least $40. Modern towboats capable of towing 6 000 tons at 5 miles per hour and having a tractive capacity of 3½ lb. per ton of cargo will cost approximately $30 per ton of cargo capacity.

The prevailing practice on a number of important coal roads provides a tractive capacity of about 20 lb. per ton of cargo. The cost of the locomotive required, with its tender, is about $1 per lb. of tractive capacity, or $20 per ton of cargo capacity.

Considering the relative speed of railroad and waterway equipment, it would seem, on first consideration, that barges and towboats, as their speed is much less, would travel a much shorter distance per year than locomotives. As a matter of fact, however, this is not true in practice, as the time used in transit is a small percentage of the total time and the less congested terminal conditions on waterways involve less terminal delays. Furthermore, barges and towboats are delayed less for ordinary upkeep service and repairs.

Freight-car performance on coal roads having short hauls is as low as 10 miles per day, while for average hauls (250 miles) it is about 25 miles per day. The cars are moving therefore only 1 or 2 hours per day, while a barge to accomplish the same result would have to work from 2 to 5 hours, an easy accomplishment by reason of lesser delays at terminals.

Actual comparison of railroad and waterway movements for comparable distances indicates that barges will make an annual mileage comparable with that of freight cars and that annual towboat mileages will be about 75% of freight locomotive mileages, the annual travel of which is about 25 000 miles.

The relation of the principal items that make up the cost of transportation both by water and by rail, exclusive of fixed charges on the thoroughfare, and an estimate of the amount of each for a specific movement is shown in Table 3. These costs presuppose a large tonnage, a steady movement, and no return cargo, the latter condition being usual for coal transportation either by rail or by water. It is assumed also for the waterway that it will be open to navigation for the full year except for such interruptions as will occur from high water. The assumed length of haul (250 miles) approximates the average haul for railroads serving the large coal fields of West Virginia and Kentucky. The costs of equipment are based on the previously stated estimates of unit costs and annual mileages. The costs of the various items under "Haulage" are consistent with actual costs obtained from a study of statistics of both rail and water transportation, those for railroad transportation having been obtained from "Statistics of Railways" for 1922, as prepared by the Interstate Commerce Commission.

A condensed statement of the railroad statistics analyzed is given in Table 4. This analysis covers important railroads whose principal tonnage is coal

or iron ore and whose passenger traffic is relatively light. Railroad transportation costs, as published in the Interstate Commerce Commission report referred to, do not distinguish between freight and passenger costs and it is necessary, therefore, in separating these costs to make assumptions which introduce some error into the costs assigned to freight transportation. This error, however, has been minimized by selecting roads having a minimum percentage of passenger service. The railroad haulage costs given in Table 3 for the first two items are obtained by taking the indicated percentage of the estimated cost of new equipment, as given in the upper part of the table, while the other items are from Table 4 for the railroad having minimum cost. If the average of the five railroads is used, the total haulage cost by rail is 4.80 mills per ton-mile instead of 3.86 mills.

TABLE 3.—Cost of Equipment and Haulage. Comparison for Railroad and Canalized River.

(Charges on Thoroughfare not Included).

Equipment.

Basis of cost of equipment.	Waterway.			Railroad.		
	Barges.	Towboats.	Totals.	Cars.	Locomotives.	Totals.
Per ton of capacity	$20.00	$30.00	$50.00	$40.00	$20.00	$60.00
Per ton of annual capacity	0.84	0.84	1.68	2.00	0.60	2.60
Per ton-mile per year	0.0033	0.0033	0.0067	0.0080	0.0024	0.0104

Haulage.

Items of operating costs.	Waterway.		Railroad.	
	Cents per ton.	Mills per ton-mile.	Cents. per ton.	Mills per ton-mile.
Interest on equipment, 6%	10.10	0.40	15.8	0.63
Depreciation on equipment*	5.05	0.20	7.75	0.31
Maintenance of equipment	10.0	0.40	33.2	01.33
Fuel	5.0	0.20	8.5	0.34
Crews	15.0	0.60	9.25	0.37
Yard and station employees	5.0	0.20	6.0	0.24
Superintendence, general expense, insurance, taxes, dispatching, loss, damage, supplies and miscellaneous	12.5	0.50	16.0	0.64
Totals	62.65	2.50	96.5	3.86

Notes.—Costs based on hauling coal 250 miles, no return cargo. Waterway: 6 000-ton cargoes in 1 000-ton barges, 9 ft. 0-in. draft, wide river, large tonnage and steady movement. Railroad: 3 000-ton train loads in 50-ton cars, favorable grades and curves, large tonnage, and steady movement.

* Calculated as sinking fund for replacement in 20 years.

For either a waterway or a railroad the ton-mile cost of interest and maintenance for the thoroughfare will vary inversely with the density of traffic. For the five railroads the costs of which are analyzed the average maintenance charge is 0.81 mill per ton-mile. The average annual traffic density of these

roads is about 4 000 000 ton-miles per mile of main track. In view of the fact that these roads have considerable double track, this is regarded as a reasonably high traffic density for roads operating both freight and passenger service.

TABLE 4.—Condensed Statistics of Important Factors Affecting Cost of Freight Transportation and Estimate of Cost of Freight Transportation on Five Railroads.

(Costs Given in Mills per Ton-Mile.)

	Railroad No. 1.	Railroad No. 2.	Railroad No. 3.	Railroad No. 4.	Railroad No. 5.	Average.
Mine products, percentage of total tonnage	92	90	83	80	79	.....
Ratio of passenger service revenue to total operating expenses, percentage.	8.0	5.2	12.0	17.0	19.4	.....
Operating ratio, percentage	65.44	69.46	77.57	75.27	79.17	73.40
Cars loaded, percentage	53.0	68.0	59.0	59.0	60.0	.....
Average haul, in miles	342	108	117	279	263	.....
Average loaded car load, in tons	57.1	49.7	45.3	41.5	41.5	.....
Average miles per day for freight cars	24.6	9.6	11.1	26.4	22.7	.....
Average tractive power of locomotives, in pounds	69 500	43 700	45 500	50 000	46 000	.....
Average train load, in tons	1 800	1 401	1 480	1 127	1 254	.....
Ton-miles per annum, in billions	2.639	1.809	1.516	11.097	10.519	.....
Ton-miles per mile of main track	4 650 000	4 760 000	3 550 000	3 825 000	3 220 000	4 001 000
Value of equipment per ton-mile						
} Locomotives*	2.19	3.03	2.93	2.72	2.29	2.6
} Cars*	6.25	12.5	12.3	7.10	7.72	9.2
(1) Interest on equipment at 6%†	0.42	0.78	0.76	0.49	0.50	0.71
(2) Interest on road at 6%†	1.94	0.88	1.01	0.89	0.80	1.10
(3) Depreciation on equipment	0.20	0.40	0.38	0.32	0.23	0.31
(4) Maintenance of way and structures	0.76	0.70	0.85	0.93	0.80	0.81
(5) Maintenance of equipment	1.33	1.55	1.98	1.50	1.63	1.60
(6) Fuel for locomotives	0.34	0.54	0.67	0.45	0.52	0.50
(7) Train crews	0.37	0.47	0.46	0.47	0.48	0.45
(8) Yard and station employees	0.24	0.44	0.91	0.52	0.54	0.53
(9) Superintendence, general expense, traffic dispatching, loss and damage, insurance, supplies	0.64	0.98	0.77	0.73	0.82	0.78
(10) Total of Items (3) to (9)	3.88	5.03	6.02	4.93	5.02	4.98
(11) Average receipts	6.70	7.76	7.84	7.45	6.86	7.32
(12) Available for Items (1) and (2) (Item (11) less Item (10))	2.82	2.73	1.82	2.52	1.84	2.34
(13) Estimate for Items (1) and (2)	2.36	1.66	1.77	1.38	1.80	1.81

* Estimated at 75% of cost of new equipment.

† From estimated valuation.

The "book value" of these roads has been published by the Interstate Commerce Commission,* but as this value includes equipment and facilities for passenger service, the valuation chargeable to freight transportation cannot be closely determined from these statistics, neither can it be said that the valuations given are equitable. However, by subtracting from the valuation estimated as assignable to freight transportation the estimated value of freight equipment, the value per mile of main track has been approximated. The interest cost per ton-mile under these assumptions has been estimated; for the five roads it averages 1.1 mills or $4 400 per mile of main track for the average annual traffic density of 4 000 000 ton-miles per mile. For a 6% return this corresponds to a valuation of approximately $75 000 per mile, a figure

* "Statistics of Railways," 1922.

which is consistent with averages for the Eastern District and with more specific cost data for roads having comparable physical features. Their average estimated charge for interest and maintenance of way is 1.81 mills per ton-mile, which figure might be reduced somewhat, under favorable conditions, for a railroad built exclusively for freight, but which is regarded as a fair average cost chargeable to the transportation of coal or similar bulk commodities.

A river well suited to canalization with modern locks adapted for coal tows may cost at present prices from $50 000 to $150 000 per mile, and such a river would have a capacity of more than 10 000 000 ton-miles per mile. The cost of canalizing the Ohio River will be nearly $110 000 per mile. In general, the cost of a canalized river will be less than that of a railroad of equal capacity, but if the traffic density is low, a railroad which will handle a light traffic may be built at a first cost less than that of improving the river and may be expanded as traffic develops, the cost eventually reaching a figure equal to or greater than that of a canalized river of equal capacity.

The Monongahela River, which is improved for a distance of 125 miles, has a traffic density in the lower 60 miles of 20 000 000 ton-miles per mile, or nearly 10 000 000 ton-miles per mile considering its entire improved length. The cost of interest, maintenance, and operation does not exceed 1 mill per ton-mile as compared with 1.8 mills per ton-mile for the five railroads in Table 4. The annual interest, maintenance, and operation charges for the Ohio River will approximate $9 000 per mile, assuming the same rate of interest (6%) as previously used for the railroad estimate. With a traffic density of 5 000 000 ton-miles per mile, the fixed charges will be 1.8 mills per ton-mile, a figure comparable with railroad costs of interest plus maintenance. The capacity of the river is from five to ten times this traffic density; if it reaches even 10 000 000 ton-miles per mile, the fixed charges carried by the Federal Government are estimated to become less than 1 mill per ton-mile.

The foregoing estimates indicate that there are possibilities in improved rivers which are well worth considering by shippers of large tonnages of bulk freight under conditions where terminal costs compare at all favorably with those of rail shipment.

H. F. Dunham,* M. Am. Soc. C. E. (by letter).—In a synopsis and paper so comprehensive and far-reaching it is difficult to discern all of the ideas and thought relations in the mind of the author. One should realize that from the first pioneer highway that connected two coast villages about 1635 to the largest modern transportation problem the effort has been unremitting "to secure an exercise of unprejudiced judgment" before making large expenditures. Efforts to this end in the distant past have been, and in the present are, hedged about with difficulties, of which three may seem worthy of further analysis than the author has given.

The element of time causes many departures from an expected and approved course. In illustration, two rival railway companies may be cited: Each determines to secure control of certain terminal rights or of locations in the

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mountains before the other can acquire them. Vast expenditures, unwarranted by the physical conditions, are then made. To say there should be no such rivalry does not prevent its existence or its appearance at any period. To say the railroads should be under different control is also futile as shown by attempts to build subways in New York, by the importance of time in Wall Street and in Washington as well, where the *Congressional Record* relates that scores of millions were saved through the defeat of a Federal waterway bill by a Congressman who continued speaking without interruption for two days more or less or until the final adjournment of that session. Truly time is a factor in the affairs of men and of governments.

The second feature, that of enhanced values, is the effect of new enterprises on real estate and other interests. Trenching for canals could have been accomplished, hundreds of miles of railroad could have been graded and the superstructure provided, all by outside parties having no other interest than that of avoiding a few years of delay in the arrival of the first canal boat or the first steam train; it would have been good real estate business in those early days, but co-operation was less common then and the men and the prairies had to wait. To-day in city planning and development, like principles and their effects may be noted.

A third element is found in the phrase "political divisions". For nearly a hundred years there have been engineers in the United States competent to locate a railway to connect St. Paul, Minn., with the Great Lakes. There were no large rivers to cross; there was the wonderful harbor of St. Louis Bay, bordered on the south by a level plateau, 40 ft. above Lake Superior—the ideal site for a city of millions. But what is found there to-day? A railway following down the tortuous left bank of the St. Louis River to a city on the side of a mountain 500 ft. high and so steep as to be hardly habitable, with sewers and water and gas mains embedded in solid granite—a city practically without harbor facilities until decades of legal and physical battle secured them. Why all this needless expense and confusion? Simply because it was set forth in the charter for a railway from St. Paul to the Great Lakes that every foot of the roadbed should lie in the State of Minnesota.

This may be called an extreme case, but it is the "extremity" in each case which shows how fundamental and unchangeable are the involved principles and how they interfere with the best of theoretical conclusions and readily formulated ideals.

William H. Burr,* M. Am. Soc. C. E. (by letter).—In discussing General Black's paper, the writer has no other purpose in view than a general engineering consideration of the subject. He does not believe that the construction or operation of a limited waterway, more or less parallel to a railroad, can seriously affect the freight business of that railroad except under such extraordinary conditions as those exemplified by the Panama Canal, or the St. Mary's Canal. In cases like the Barge Canal in the State of New York, it is feasible for a limited waterway to stimulate the growth of a profitable business without trenching on the established business of a parallel railroad. In order to do

* Cons. Engr., New York, N. Y.

that, however, the carriers must be so designed and built, and be so equipped with power, as to meet the economic requirements of the situation. Unfortunately, neither the Federal Government, nor a State Government, has yet shown evidence of the requisite business capacity to administer successfully such an undertaking, in so far as the public interests may require.

General Black's paper sets forth some of the fundamental elements of limited waterway transportation which have not heretofore been so well reduced to a quantitative basis.

All the elements of railroad transportation have been known and computable, so to speak, for years, but water transportation in limited waterways has not been subject to such quantitative treatment. In other words, canals and similar limited waterways have been fitted with boats designed and built to meet purely local conditions without regard to economic principles, either as to transportation resistances, or as to the economic effect of the dimensions of the waterway on transportation costs. One of the fundamentally valuable features of this paper is the setting forth of economic elements of the broad problem in such manner that a limited waterway transportation project may receive the same analytic consideration as a railroad project.

As the first result of these considerations, apparently the author would make the decision as to whether a transportation project should be approved and actually constructed by the aid of its purely analytic features. At the very outset of the paper, he states:

> "New waterway lines of transportation should not be established unless an analysis of the existing railway, highway, and waterway lines and of the need for transportation shows that the proposed new line is required and that if established it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs."

To base the inception and actual construction of a railroad on the estimated profits which it may earn, is, of course, an old method of procedure, although in the early days of railroading it was not easy to predetermine by any well-defined considerations of cost, or contemplated business, whether or not a road would pay. In fact, it has been said many times that the proper time to build a given railroad line was whenever sufficient money could be raised to do it. As a consequence, many new railroad lines were built before it was possible to secure a profitable business. The individuals who placed their money in the project lost at least part of that which they expended, but the general wealth of the community was increased.

As a broad proposition, therefore, the building of the railroad was justified by the favorable effects on the community as a whole, although the effect on some individuals was ruinous.

Similarly, if the attempt should be made to control the construction of waterway transportation by the same method of estimating costs and profits in order to determine whether or not the project should be undertaken, it would be found impossible in many and perhaps most cases to predict, first, how much business would result and, second, what the actual transportation costs

would be after the transportation lines were built. In fact, there would undoubtedly be the same result as in the case of so many railroad projects.

Again, a complicating question would undoubtedly arise, especially under present conditions, in consequence of limited Federal control of the transportation business. Who would constitute the deciding body as to whether the conditions were sufficiently encouraging to undertake the project? Such a body would have a highly complicated problem to solve. The cost of transportation, as indicated by the author, would involve the costs required to overcome the resistance of the water against the moving boat, a resistance varying with the surface and form of the boat and the dimensions of the waterway, also with the kind and cost of power as well as the speed of the boat, and with other elements of the problem less adapted to definite treatment.

In addition to the various elements of cost there would be that of the actual construction which might be reasonably determinable in some cases and quite impossible of accurate determination in others. While such elements of the broad question may be, and are indeed, subject to some degree of definition, they certainly cannot be determined in advance within any such close limits as to make a reasonable basis for determination of profits.

There is, however, another source of uncertainty of the highest importance, and that is the probability of securing the requisite capital to make the project a reality. No ascertainment of cost of any kind, whether of construction, of equipment, or of transportation, can be considered sufficient for bringing success to the project unless the owners of the necessary capital are convinced that the project will be successful from a business point of view. In fact, that is the last and principal condition on which the project can be put through successfully. It would matter little what any lawfully organized body, or even disinterested experts, might decide. In fact, it comes down to the old criterion under which railroads and other projects have been carried through; if capital can be interested the project may be realized, but not otherwise. It is, therefore, scarcely feasible to subject the existence of future extension of the business of transportation in limited waterways to the test of whether or not they can be built and operated profitably. If one stops to realize that what communities are looking for, after all, are those developments which will increase their total wealth, carrying with it, on the whole, individual wealth, it will be seen that any test more or less artificial in its nature, and thus indeterminate, is both unnecessary and in reality unadapted to secure desirable results.

The résumé given by the author of formulas and authorities relating to channel dimensions, types of carriers, and features of propulsion of boats, as well as to costs of transportation, is excellent and useful. It is not to be supposed that it indicates a complete solution of problems undertaken by the various authors named. The results are far indeed from that. They do indicate, however, a rational beginning with sufficient data on which to base at least some approximate conclusions as to the design of boats, the relation of their cross-sections to that of the waterway, the amount and character of some resistances, and the variable elements on which the amounts of those resistances rest.

Other experimental investigations of the same character will doubtless continue to be made from time to time, constantly improving designs and correspondingly reducing resistances as well as furnishing data for the improvement in power. There is no reason why all these variable elements of limited waterway transportation should not ultimately be as well controlled as the elements of railroad transportation. Such ends must be attained if canal transportation is to come to its own in the United States or any other country.

Whenever the Barge Canal of the State of New York attains the position of a carrier of respectable tonnage, all the data available will be required to design proper boats, effective motor power, and to determine at least approximate resistances. This waterway, having a minimum width of bottom of 75 ft., and in the canal proper a depth of 12 ft., but with a depth of 14 ft. over the lock sills, should have an annual capacity of 20 000 000 tons, whereas during the canal season of 1924, its total traffic was only 2 032 317 tons, the highest amount reached since its completion. The commodities showing the greatest tonnage this season were petroleum and other oils, paper and paper products, and wheat. The tonnage of local traffic was 520 525, about one-quarter of the total. On the whole, such boats were used as could be found; however, the Standard Oil Company and other companies used special boats adapted to their purposes.

The total expenditures made by the State of New York on this project from the inception of the old Erie Canal to and including the Barge Canal in its present condition, have been approximately $220 367 600. Before the Erie Canal was made free, however, in 1882, it had earned and paid back to the State of New York the net sum of $93 342 000, leaving the net total expenditure at the present time of $127 025 600. This looks discouraging for canal business, but there probably never was a great business project treated with so little business wisdom, with scant consideration of any of its real problems, or with such absence of intelligent interest as this great transportation property. After the canal itself had been well constructed, it was, figuratively speaking, treated with silent contempt and neglect, and has gradually come to be regarded as about the strongest argument available against any attempt to make a limited waterway successful, even under the most favoring conditions.

As a matter of fact, the Panama Canal has been a phenomenal success and will always remain so, likewise the St. Mary's Falls Canal. Although those canals are, of course, extraordinary cases, the general proposition holds that limited waterways, equipped with properly designed boats and suitable power and apparatus for handling cargo at terminals, should be classed as among those effective means of transportation on which the growth of communities is based.

William M. Black,* M. Am. Soc. C. E. (by letter).—In a long membership in the Society, it has been the writer's experience that the discussion brought out by a paper is frequently of equal or greater professional value than are the ideas presented by the author. Certainly the discussion on this paper is no exception to the rule, even though the points discussed are quite different from those anticipated.

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It is evident that the writer was not entirely clear in his endeavor to formulate a criterion by which the economic necessity for a new transportation line should be judged. In the cases cited in the discussions, notably the improvement of the River Clyde, and the construction of the Manchester Ship Canal and of the Panama Canal, the benefits accruing from the completion of the project resulted directly from the creation of improved facilities for waterway transportation. It is equally true that pioneer work has been done in opening new lands for settlement, or in enabling National resources of timber or minerals to be utilized, by the creation of transportation facilities by highways, railways, or waterways; but in all these cases the economic advisability of the works was carefully investigated before their inception and the costs were weighed as against the expected pecuniary returns. Even the old policy of European rulers of building railways along lines of strategic importance from a military, rather than from a commercial, standpoint was based on a supposed ultimate economy to the respective States. But are there not examples in the United States where, in past years, enthusiasts have succeeded in having large sums expended for transportation lines of all classes, which were not economically justifiable? Have efforts to this end yet ceased?

The discussion on the relative merits of waterway and railway transportation is interesting and figures of value are presented; but does any one believe that each transportation medium has not its own economic sphere in the vast movements of natural and manufactured products required in the nation? Should not the question under consideration be how the total expense of the movement can be reduced, rather than the question of the earnings of this or that company? If, as on the Monongahela River, the use of the waterway diminishes the cost of production on a large proportion of the total output of one of the most necessary manufactured products, is not the required expenditure of public funds justified? In fact, the writer has been informed by one of the greatest producers of the Pittsburgh District that it could not have fulfilled the duty imposed on it and have done its allotted share of the work of the World War, had it not been for that improved waterway.

The transportation system of the nation is by no means economically perfect. Traffic does not follow natural lines; the railroad rate schedules are confessedly unscientific and needlessly cumbersome, causing discrimination against, and actual injustice to, many communities. The waterways have been developed only in parts, and the question of the economic adaptation of the carriers to the waterway channels has been too little studied. The highway system is new. The "common carrier" laws have an unequal application. The true co-ordination of all the transportation media is yet a dream of the future.

It is the belief of the writer that the full economic use of the highways, railways, and waterways can be achieved only when transportation is effected through duly supervised transportation companies and not, as to-day, by competing and more or less hostile railway companies, auto-truck companies, and steamboat and barge lines.

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REPAIRING LEAKS IN A DRY DOCK BY GROUTING*

By W. Mack Angas,† Esq.

With Discussion by Messrs. F. R. Harris, Walter Ludlow Taylor, and W. Mack Angas.

Synopsis

Serious leakage which was eroding the foundations of the 1 022-ft. dry dock at the Philadelphia Navy Yard, was successfully stopped in 1921 by the pneumatic grouting process. First, a casual inspection of the drainage culverts in the floor of the dock was made, which showed the leakage to be considerable in quantity and deeply colored with suspended matter. However, this color was not accepted as conclusive proof of serious erosion as the dock was built on a stratum of red clay which, it was believed, would give a relatively high color to water even in minute quantities. Furthermore, there had been no appreciable settlement of the structure after the slight initial movement during construction. When in the course of investigation it developed that infiltration of silt was taking place at an alarming rate, the scope of the undertaking was broadened to include the determination of a method of stopping the leakage. Grouting by the pneumatic process while the pressure below the dock was relieved through a vent in the floor, was decided on as the most practical method of repairing the dock.

The repair work consisted not only of stopping leaks through construction joints, but also of closing a fissure in the clay stratum that was feeding the leaks, and of filling a cavity which had been eroded under the floor of the dock. In spite of some unforeseen difficulties which necessitated slight deviations from the original plan of repair the process eventually proved successful.

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The dry dock at the Philadelphia Navy Yard, one of the largest structures of its kind, is situated in the western part of League Island, Philadelphia, Pa. It rests on a stratum of red clay overlying water-bearing sand. The dock had never been tight; this led to an investigation during the summer of 1921 to determine whether or not the quantity of silt and other fine material being washed into it, was great enough to jeopardize the structure and if so what measures should be taken to correct these conditions.

Investigations

Character of Inflow.—A most careful inspection of the dock preceding the investigation of the rate of infiltration of silt, disclosed three general classes of leaks. The most important consisted of those carrying silt, generally from the transverse construction joints in the dock floor into the drainage culverts. As an experiment one of the larger of the leaks was caulked, using cypress wedges, with the result that the leakage promptly appeared on the dock floor through the same construction joint and, later, when caulking of the joint in the floor was commenced, it persisted through the construction joints in the side-walls. This indicated that very little in the way of permanent repair of the dry dock could be expected by caulking, because practically every joint in the dock would ultimately have to be caulked as the water was forced to seek more and more tortuous channels. There was also a strong probability that the caulking material would work out in time. A few silt-bearing leaks occurred at cracks through which the leakage reached the floor of the dock, but these were small.

The second general class of leaks consisted of flows of ground-water from the strata of gravel and sand overlying the red clay. These leaks occurred through construction joints in both the floor and side-walls; they were not silt-carrying, but were characterized by an acrid odor peculiar to water from the strata lying between the river silt of League Island and the blanket of red clay on which the dock rests.

A third class of leaks, minor in importance, consisted of river water passing through construction joints from the filling culverts under the full river head causing inflow into either the interior of the dock, one of the drainage culverts, or the pump-well. An examination of the sanitary sewers in the floor of the dock provided for the use of ships in dock (Fig. 1) showed that they were full of silt-laden water under a considerable head. Being useless, therefore, they had been blanked off from the sewage pumps in the pump-well. When any one of the bronze caps were removed, with which the sewer risers to the dock floor were fitted, the riser discharged a brisk "full bore" stream of silt-laden water on to the dock floor.

Quantity of Leakage.—The rate of flow from the individual leaks was next determined by diverting the water into a measuring receptacle of some kind, these receptacles varying in size from a 1-gal. bucket to the rudder pit of the dock, which was used to measure the harmless leakage passing under the caisson at the dock entrance.

The sum of the flows from the individual leaks was checked against the total leakage by installing weirs in four of the drainage culverts where they

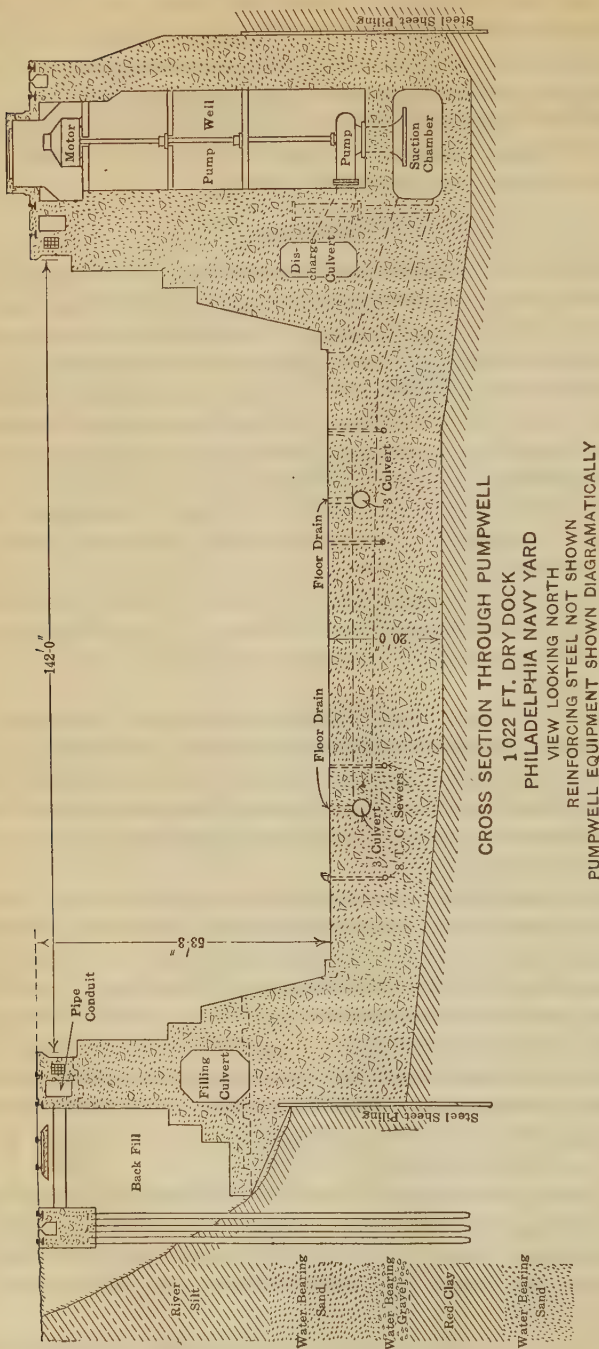

Steel Sheet Piling
Motor
Pump Well
Pump
Suction Chamber
Discharge Culvert
Floor Drain
3' Culvert
20'0"
Floor Drain
3' Culvert
R. C. Sewers
142'0"
58'8"
Pipe Conduit
Filling Culvert
Back Fill
Steel Sheet Piling
River Silt
Water Bearing Sand
Water Bearing Sand & Gravel
Red Clay
Water Bearing Sand
CROSS SECTION THROUGH PUMPWELL
1022 FT. DRY DOCK
PHILADELPHIA NAVY YARD
VIEW LOOKING NORTH
REINFORCING STEEL NOT SHOWN
PUMPWELL EQUIPMENT SHOWN DIAGRAMATICALLY
0 10 20 30 40 50
Scale in Feet

FIG. 1.

discharged into the drainage pump pit, and closing the other culverts so that the entire flow to the pumps passed over these weirs. At first, a somewhat crude type of weir was used, but this was replaced later by weirs carefully made from boiler plate, with machined crests, for which measurements of head were obtained with hook-gauges. Readings were taken at frequent intervals each day for 30 days. No appreciable change occurred in the rate of flow between high and low tide, or between the beginning and end of the 30-day period. The total leakage, as shown by the weirs, was as follows:

Weir No. 1	96.0	cu. ft. per min.
" No. 2	12.0	" " " "
" No. 3	28.2	" " " "
" No. 4	1.8	" " " "
Total	138.0	cu. ft. per min.

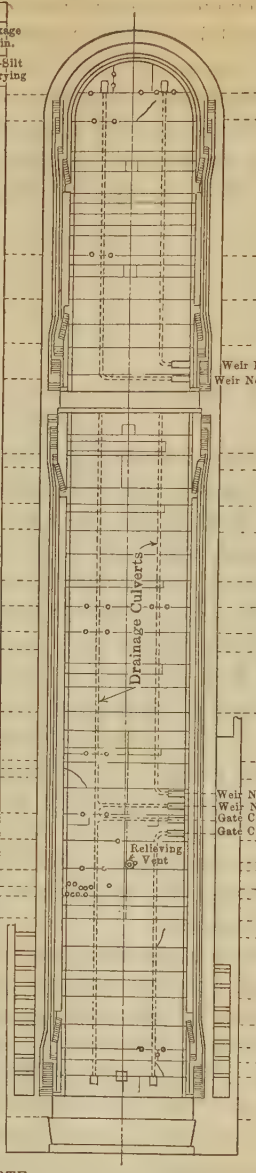
This figure closely checked the sum of the individual leaks, which was 141.8 cu. ft. per min.

The data obtained from the inspection were plotted on a plan of the dock, Fig. 2, with notations as to (1) the character of the flow from each leak; (2) the rate of flow; and (3) the exact location of the construction joint, crack, or other orifice from which the water was issuing. Data on the subsequent grouting operation also appear on Fig. 2.

Soil Content in Leaks.—An attempt was made to determine the quantity of solid matter being carried into the dock by each leak. For this purpose, samples of water from the various leaks were taken at frequent intervals and the percentage of solids in them was determined in the Chemical Laboratory at the Navy Yard. The results obtained were erratic, probably being influenced by the exact position of the receptacle in which the sample was caught with relation to the orifice through which the water was flowing. However, results that were consistent and, therefore, of value, were obtained by sampling the flow over the weirs in the drainage culverts each day for a month and thus determining the total rate of infiltration into the dock rather than that due to any one leak.

The method used in determining the percentage of solids was, as follows: A 1-gal. sample of water was filtered through filter paper of a character that left a low percentage of ash when burned (such paper as is commonly used in quantitative analysis). The material remaining on the paper after filtration was then dried at a temperature of 100° to 105° cent. until no more moisture could be driven off. Two small quantities of the dried material were removed, and the weight and specific gravity of one of these small samples were determined in much the same way as for Portland cement. The other sample was weighed, heated to incandescence, and again weighed to determine the loss due to ignition. The filter paper with the major part of the residue was then heated to incandescence, the paper itself becoming ash of negligible quantity, and the material then cooled and weighed. The correction factor previously obtained for the loss due to ignition was then applied to the entire weight of the ignited residue to give the total weight of the dry material suspended

CONDITION BEFORE GROUTING				RESULT OF GROUTING		
Station	Location and Description of Leak	Quantity Leakage Cu. Ft. per Min.		Quantity of Grout Placed, Cubic Feet	Quantity Leakage after Grouting	
		Silt Carrying	Non-Silt Carrying		Silt Carrying	Non-Silt Carrying
9+72	Equal Leaks into East and West Culverts and Floor	4.0	0.0	70.4	0.0	0.0
9+45	0.5 Cu. Ft. through cracks from Sewer, 0.3 Cu. Ft. into West Culvert	0.8	0.0	272.2	0.0	0.0
8+21	Into West Culvert through Construction Joint	1.5	0.0	75.2	0.0	0.0
7+81	Through 2 in. Pipe from West Wall to West Culvert	0.0	3.0		0.0	3.0
7+41	Through 3 in. Pipe from West Wall to West Culvert	0.0	6.0		0.0	6.0
7+08	Into Cross-Culvert, 35 Ft. East of Center Line	6.0	0.0	Weir No. 4 Weir No. 3 (* See Note)	0.0	0.0
6+89	To Floor through Construction Joint, West of Center Line	0.1	0.0	*	0.0	0.0
6+27	To Floor through Construction Joint	1.0	0.0	*	0.0	0.0
5+92	To Floor through Longitudinal Construction Joint	0.2	0.0	*	0.0	0.0
5+68	Trace through Construction Joint to Floor	Trace	0.0	*	0.0	0.0
5+22	Through Construction Joint to Floor	0.5	0.0	*	0.0	0.0
4+98	To Culverts through Construction Joint	16.0	0.0	60.8	0.0	0.0
4+73	Through Construction Joint to West Culvert	1.5	0.0	123.0	0.0	0.0
4+49	3-in. Pipe from West Wall and through Construction Joint to Floor	0.6	6.0	*	0.0	6.0
4+00	Trace through Construction Joint to Floor	Trace	0.0	*	0.0	0.0
3+54	Through Construction Joint to West Culvert	1.8	0.0	280.0	0.0	0.0
3+50	Through 3 in. Pipe from West Wall	0.0	8.0		0.0	8.0
	Crack Leaking Trace	Trace	0.0	(Caulked)	0.0	0.0
3+06	Through Construction Joint to West Culvert	17.0	0.0	Weir No. 2 Weir No. 1 Gate Closed 104.0	0.0	0.0
2+82	Through Construction Joint to Floor	8.0	0.0	Gate Closed 127.2	0.0	0.0
2+60	Pressure Relieving Vent			6895.8	0.0	0.0
2+58	Through Construction Joint into West Culvert	3.3	0.0	160.0	0.0	0.0
2+36	Through Transverse Crack into Culvert Shaft West Side	5.8	0.0	737.6	0.0	0.0
2+32	Through Construction Joint to Floor West Side	0.1	0.0	*	0.0	0.0
2+09	Through Construction Joint to West Culvert and Floor	0.7	0.0	*	0.0	0.0
	Through Crack to Floor	0.1	0.0	*	0.0	0.0
1+37	Through Construction Joint to West Culvert	0.3	0.0	478.4	0.0	0.0
1+18	Through Construction Joint to West Culvert	0.2	0.0	*	0.0	0.0
0+90	Through Construction Joint to Culverts and Floor	1.3	0.0	*	0.0	0.0
0+84	Through Construction Joint to Wall from Filling Culvert	0.0	20.0		0.0	20.0
0+80	Through Construction Joint to West Culvert and Floor	0.3	0.0	92.2	0.0	0.0
0+65	Through Construction Joint to Rudder Pit	0.2	0.0	*	0.0	0.0
0+25	Leakage past Entrance Caisson	0.0	28.0		0.0	28.0



PLAN
1 025-FT. DRY DOCK
PHILADELPHIA NAVY YARD
SHOWING
LEAKAGE BEFORE AND
AFTER GROUTING

NOTE:

Grout Holes shown thus ∘

A Star (*) in the column of Grout Quantities indicates that Grout worked along to Leak stopping it and rendering specific Grouting of the opening unnecessary.

Construction Joints in Floor appear as fine solid line.

FIG. 2.

in the water. From the weight and specific gravity of the material, its volume —the net volume of the solids, neglecting voids—was found.

Using the average percentage of solids in the flows over the various weirs for a month, the situation with regard to the rate of erosion under the dock was as given in Table 1.

Foundation Erosion.—The specific gravity of a representative sample of the red clay on which the dock rests, had been determined while the excavation was in progress. From this and the specific gravity of the dry material, the percentage of voids in the clay was computed as 17. Therefore, according to the best available information, the gross volume of solids carried into the dock was 38.85 cu. ft. per day. That this figure was probably somewhat in error was fully realized; the most doubtful factor in the computation, but by no means the only doubtful factor, was the assumption that the voids in all the eroded material averaged 17 per cent. The investigation thus far had shown, however, that solids or solid materials were being carried into the dock at a rate of from 1¼ to 1½ cu. yd. per day, with the possibility that the rate was as high as 2 cu. yd. per day, and the certainty that it was at least 1 cu. yd. per day. This erosion could not continue unchecked without ultimately seriously injuring the dock, or destroying it.

TABLE 1.—Erosion from Leakage.

Weir.	Flow, in cubic feet per day.	Average percentage of solids for a month.	Net volume of solids, in cubic feet per day.
No. 1	138 240	0.017	23.50
" 2	17 280	0.023	3.97
" 3	40 608	0.011	4.47
" 4	2 592	0.012	0.31
Total	.....	.....	32.25

It was impossible to determine even roughly how long erosion had been going on at this rate, or to what extent it had progressed under the dock; but it was evident that a large quantity of subsoil material on which the dock rested had been washed out, probably resulting in large cavities under the floor of the dock. That the floor had apparently not settled indicated that the cavities might consist of numerous small watercourses, instead of a few large openings. Rough approximations gave the extent of these cavities as several hundred cubic yards, one estimate being 600 cu. yd. These approximations had some bearing on the methods eventually selected for grouting the leaks and cavities, as they indicated the necessity of having on hand, before the commencement of grouting, appliances and materials for forcing several hundred yards of grout under the dock, in order to accomplish satisfactory repairs. A study of the construction records of the dock completed the investigation.

Construction Records.—During the building of the dock, water encountered in the strata of gravel and sand overlying the red clay on which the dock

floor rests, had given such serious trouble as to cause radical changes in the original method of construction and, by the time excavation had progressed to approximately 27 ft. below mean low water, necessitated modifications in the design of the dock. Similarly, water from a stratum of fine white sand underlying the red clay had given serious trouble wherever a fissure in the clay let the water and sand flow through.

The original scheme of construction had contemplated doing practically all the excavation with a suction dredge, working inside a coffer-dam across the dock entrance when the work had progressed so far as to require lowering the water in the excavation below river level. In this manner, good progress was made until a depth of from 27 to 30 ft. below mean low water was reached, when the sides of the excavation started to slide.

An investigation showed the trouble to be due to an inrush of water from the sand and gravel lying below the river silt that forms League Island. It was found impractical to carry the suction dredging farther; as fast as more bottom excavation was removed, the banks slid in, because of the sand being washed out from under them, and it was impossible to maintain reasonable side slopes. Numerous core borings taken on the site before the work was commenced, had shown the existence of the stratum of clay on which the dock was to rest, but had not indicated that serious trouble with water was to be expected.

At this point, the dredge was removed from the excavation, the coffer-dam across the dock entrance again closed, and, after a lapse of about a year spent in investigation and design, excavation was re-commenced "in the dry". The entire excavation was surrounded with steel sheet-piling driven well into the red clay in order to prevent any flow of water from the stratum of gravel, and the large area thus surrounded was subdivided by transverse rows of steel piling spaced from 300 to 400 ft. apart. The sheet-piling was braced by plumb and batter piles of timber. As it was considered unwise to risk a large and deep opening, the excavation was made in sections just long enough to permit the placing of one strip of floor at a time, each being the full width of the dock, but varying in length from 10 to 30 ft. As soon as a section of flooring had set, the excavation was widened to accommodate the adjacent sections of side-wall foundation which were poured as soon as possible. The steel sheet-piling at the sides of the excavation was left in place (Fig. 1). A detailed description of the methods of construction and the manner of handling the heavy shoring required to brace the sheet-piling, has already been published.*

During the entire course of the construction of the floor and the foundations for the side-walls more or less trouble occurred with water getting through the sheet-piling along the sides of the excavation at points where the piling had been injured by striking boulders in the stratum of gravel overlying the red clay. The excavation was kept dry by constant pumping with centrifugal pumps having a combined capacity of from 5 000 to 8 000 gal. per min. To combat leaks through the sheet-piling small auxiliary coffer-dams were con-

* *Engineering News-Record*, April 15, 1920.

structed outside and, in these, pumps were installed to intercept the flow of water before it reached the main excavation. In some cases pipes were placed in the floor of the dock to conduct harmless leakage from such coffer-dams to the drainage culverts.

Leaks occurring through fissures in the blanket of red clay proved much more difficult to handle. The procedure was to build over the leak a heavy timber crib with an opening toward the dock entrance of sufficient size to permit the entire flow to escape without building up pressure within the crib. The leakage was then piped from this opening toward the dock entrance into temporary sumps from which it was pumped out of the excavation; the pipe was extended and the sumps were moved toward the dock entrance as the construction of the floor progressed. After the floor had been placed for 200 or 300 ft. beyond the leak, the pipe was plugged and the leakage thus stopped.

This method, although satisfactory in the case of several small openings, did not meet with unqualified success when applied to the large fissure in the red clay at Station 2 + 60 from the entrance caisson seat, a little to the east of the center line of the dock. After this leak had been covered, and when almost enough flooring had been placed to warrant plugging the discharge pipe, the floor of the dock in the neighborhood of the leak settled slightly, shearing the discharge pipe at the construction joints and, as it developed later, shearing the terra-cotta sewer pipes at the same points. The settlement was more pronounced at Station 2 + 36 than at the fissure itself, and there was no means of telling whether it was due to the erosion of material from beneath the clay blanket by the leak, or whether it was simply from the natural compacting under the weight of the floor and side-walls. No leakage appeared on the floor of the dock and it was thought that no further trouble would occur when the discharge pipe from the leak had been plugged, pumping discontinued, and the dock completed by the construction of the caisson seat. A layer of concrete was placed on the sections of floor that had settled so as to bring the surface to grade.

The fact that all the serious leakage into the completed dry dock was silt-bearing and of the same general character as the flow from the fissure in the red clay at Station 2 + 60, indicated that this fissure was probably the source, and that water from it was working its way under the dock to the construction joints, through which it entered the drainage culverts and, in a few instances, appeared on the floor. There also seemed to be a strong probability that the 8-in. terra-cotta sanitary sewers in the lower part of the dock floor were carrying leakage along from one construction joint to another, as settlement of the floor had undoubtedly sheared or cracked the terra-cotta pipes at the construction joints and, as has been stated previously, both sewer pipes were filled with silt-laden water under a considerable pressure. Therefore, the plan for repairing the dock was based on the assumption that all the silt-carrying leakage originated at the fissure in the red clay at Station 2 + 60.

Methods for Repairs

Two general methods of grouting the leaking construction joints in the dry dock were considered. The first of these, which had proved moderately

successful in stopping small leaks through the floor where the most serious settlement had taken place, was to insert grout pipes into construction joints and leaks in the floor when the dock was empty, to extend the pipes to the coping of the dock, and to connect them to a compressed-air grouting machine of the usual type. The dock was then flooded and grout was forced through the long and necessarily somewhat tortuous piping. The great advantage of the method was that it assured proper conditions under which the cement could set, because with the dock flooded pressures inside and outside were equalized and there was no flow to wash out or disturb the grout. The method, however, was open to two serious objections: First, it would necessitate the frequent and expensive flooding and unwatering of the dry dock as the work progressed; and, second, the long lines of pipe leading from the coping to the floor of the dock would be likely to become choked before the cavity had been satisfactorily filled with grout.

The second method, which was adopted, contemplated drilling a large hole through the floor of the dock into the big fissure in the clay blanket at Station 2 + 60, thus allowing the leakage to enter the dock freely through this artificially made relieving vent and reducing the flow through the construction joints and other leaks so that they could be grouted with the dock empty. Those leaks most remote from the fissure in the clay were to be grouted first, progressing from both ends toward the fissure, until the entire dock was tight with the exception of the relieving vent. Grout pipes were then to be connected to this relieving vent and led to the coping, the dock was to be flooded, and any cavity which might exist below the vent, was to be grouted with machines placed on the pavement at the side of the dock.

Repairing Leaks

Drilling.—The actual work of repairing the dock was started by drilling the relieving vent through the floor of the dock over the fissure in the red clay. An ordinary portable Artesian well-drilling rig was used and did the work quickly and effectively. In Fig. 3 is shown the outfit at work on a grout hole, after the relieving vent had been finished. Details of the relieving vent are shown in Fig. 5, together with the arrangement of grout pipes used later for filling the cavity under the floor.

The hole through the floor of the dock was started with a 12-in. drill and carried to a depth of 5 ft. with this tool. An 8-ft. length of 10-in. steam pipe was then inserted into the hole and securely fastened with a caulked lead joint. At its upper end it was equipped with a flange and valve so that the flow through the floor could be controlled when the drill broke into the fissure below. Drilling was then resumed with an 8-in. tool and carried through the dock floor and the timber crib which had been built over the fissure in the clay. At first the flow was disappointingly small, but it soon increased to the quantity shown in Fig. 4. An examination of the various leaks in the dock showed that those near the caisson stopped flowing entirely when the 10-in. valve on the vent was opened and all the others were greatly reduced in volume. There was every indication that the proposed grouting program

Fig. 3.—Artesian Well-Drilling Outfit Used for Drilling Relieving Vent and Large Grout Holes.

Fig. 4.—View of Relieving Vent, Showing Natural Flow from Vent Previous to Installation of Pump.

could be carried out successfully; accordingly, preparations were pushed forward for the next stage of the work.

A rock drill was used for drilling grout holes into construction joints and cracks. This work was slow as the drills frequently jammed. Fig. 2 shows the locations of all the grout holes and the manner in which they were placed in pairs at opposite sides of the drainage culverts to insure the grout working completely around them. In Fig. 5 is shown a typical grout hole near the relieving vent, also the method by which a piece of 2-in. pipe was fastened into the concrete with a caulked lead joint to permit the attachment of the hose from the grouting machines.

The holes were carried almost through the floor of the dock, as shown (Fig. 5). Those near the caisson were drilled first; the drill gang was then

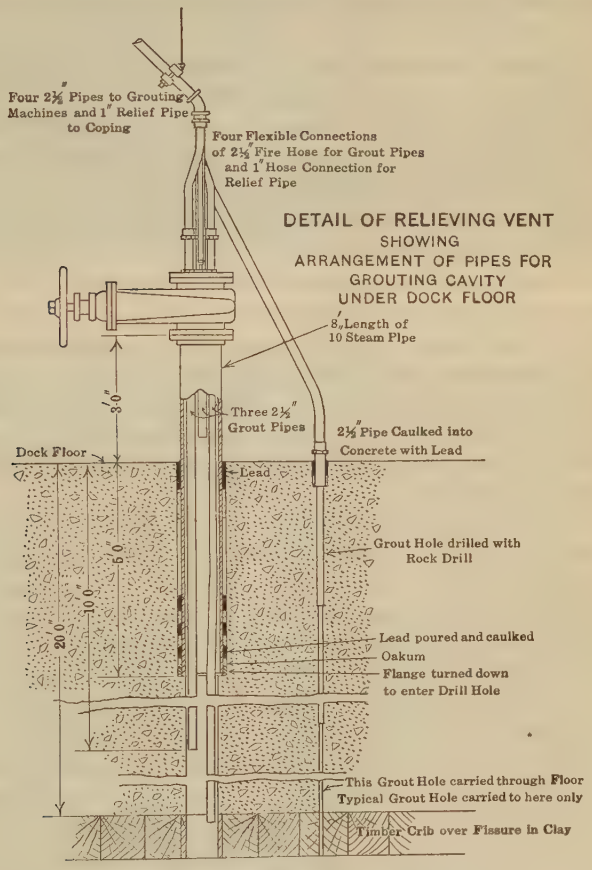

Four 2½" Pipes to Grouting Machines and 1" Relief Pipe to Coping
Four Flexible Connections of 2½" Fire Hose for Grout Pipes and 1" Hose Connection for Relief Pipe
DETAIL OF RELIEVING VENT
SHOWING
ARRANGEMENT OF PIPES FOR
GROUTING CAVITY
UNDER DOCK FLOOR
8' Length of 10 Steam Pipe
30"
Three 2½" Grout Pipes
2½" Pipe Caulked into Concrete with Lead
Dock Floor
Lead
Grout Hole drilled with Rock Drill
50'
10'
20'
Lead poured and caulked
Oakum
Flange turned down to enter Drill Hole
This Grout Hole carried through Floor
Typical Grout Hole carried to here only
Timber Crib over Fissure in Clay

FIG. 5.

moved toward the relieving vent, preparing each leaking joint for the grouting gang which followed closely. In many cases the grout worked along under the dock a number of feet, closing other leaks (as noted in Fig. 2). When the vent was reached a number of closely interconnected leaks near-by were

grouted and the gangs were then moved to the head of the dock, from which they again started working toward the vent.

All grout holes were drilled with the rock drill except two at opposite sides of the west drainage culvert in the construction joint at Station 3 + 06. This leak was particularly large and the grout holes were consequently bored with the Artesian well-drilling outfit, using a 6-in. bit, so that there would be no question about securing a satisfactory channel for the grout to reach the bottom of the construction joint. These holes also differed from the others in that they were carried completely through the floor of the dock.

Mixing and Placing Grout.—The experience of the contractor in grouting the leaky section of floor at Station 2 + 36 indicated the necessity of maintaining an almost constant flow of material to prevent its setting in the grout pipes. Thereafter, three grouting machines were mounted as a battery on a heavy timber platform which could be readily moved by the dry dock cranes. As shown in Fig. 6, there were two vertical machines of the type that mix the grout by agitating it with air admitted at the bottom of the tank, causing the contents to "boil"; the third machine used a paddle-wheel driven by a compressed-air engine. All three machines worked well throughout the job.

It was found necessary to use neat cement in grouting small cracks while an equal mixture of Portland cement and fine building sand was used in the larger cracks. Grout was mixed to the consistency of thin gruel. The training of the crews to co-ordinate the operation of the three machines was the most difficult part of this work. When once the men "got the hang of the thing", the work went smoothly and with very little interruption from choked grout pipes. Compressed air at full yard pressure—100 lb. per sq. in.—was used in forcing the grout into the leaks. Lower pressures were also tried, but they led to frequent trouble with choked pipes. Soft wood wedges were used to caulk joints in the drainage culverts temporarily and on the dock floor to prevent the escape of grout.

Overcoming Difficulties Near Station 2 + 36.—Grout placed in three locations near the caisson seat by the methods described completely stopped all silt-carrying leakage in the outer 200 ft. of the dock. It was believed that the technique of the work had been so thoroughly developed during these first three operations that no further trouble would be experienced in finishing the repairs.

The job did not prove quite so simple, however. While a night shift was grouting leaks in the west side of the dock, near Station 2 + 36, a big crack opened in the floor through which a large volume of water entered, notwithstanding that the relieving vent was wide open. The recently placed grout was washed out in a few minutes, and after four or five hours spent in futile attempts to caulk the new crack with big wooden wedges, work was suspended until daylight. A careful examination revealed a serious situation—the dock floor was found to have lowered several inches and to have cracked where settlement had occurred during construction. In sinking, however, the main floor had pulled away from the thin upper layer of concrete which had previously been added to bring the floor to grade. The new settlement and crack-

FIG. 6.—THREE GROUTING MACHINES MOUNTED AS A BATTERY ON TIMBER PLATFORM IN ORDER TO BE MOVED READILY BY DRY DOCK CRANES.

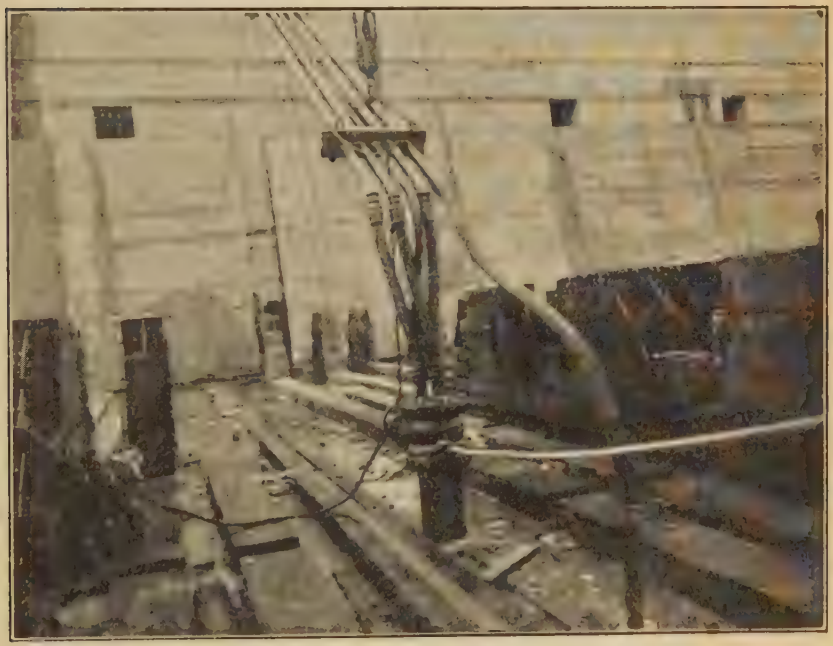


FIG. 7.—CONNECTIONS TO RELIEVING VENT FOR GROUTING CAVITY BELOW FLOOR OF DRY DOCK.

ing of the floor, therefore, had not been noticeable until the grout worked its way between the floor proper and the thin top layer, thus forming a sort of blister which finally burst and showed the true state of affairs.

The top layer of concrete was completely removed to make a thorough inspection possible. Nothing in the way of repairs could be accomplished until the flow of water through the crack was stopped, and the only practical method of accomplishing this was to increase the flow from the relieving vent. A 6-in. electrically driven centrifugal pump, therefore, was connected to the vent to increase the flow. Originally, wire strainers were installed between the 10-in. valve on top of the vent and the pump, but they were removed when found to clog with lumps of clay that readily passed through the pump. With the pump running, there was no leakage on to the floor through the newly opened crack and very little into the drainage culverts. Grouting was resumed, therefore, and carried on without further mishap.

A number of grout holes were drilled in the floor of the dock in this area where settlement had taken place so as to intersect the settlement crack and permit its being completely filled with grout. Some old grout pipes left by the original contractor were also uncovered when the false floor was removed and one or two that did not seem to have been thoroughly filled were opened and grouted again. The settled section of floor was then brought to grade with concrete. At this stage of the proceedings, the terra-cotta sewers in the dock floor were blown free of silt and then grouted, as they were evidently the channel through which much leakage was traveling up and down the dock from the fissure at Station 2 + 60. In all, more than 1 000 cu. ft. of grout was placed in the west side of the dock floor between Stations 2 + 32 and 2 + 82 before the leakage was stopped.

Completion of Grouting of Leaks.—The leaks between Stations 2 + 60 and 3 + 04 were grouted at the same time—a deviation from the original plan of progressing from both ends of the dock toward the relieving vent. This was done because there were many indications that they were closely connected and could be advantageously repaired together. When all these leaks had been stopped, and the grouting had held for a few days with the relieving pipe closed, the machines were taken to the head of the dock and the remaining leaks were grouted following the original plan.

An examination of the dock made after the grouting was completed and with the relieving vent closed, showed that all silt-carrying leakage had been stopped, but that the clear side-wall leakage had not been affected. The total flow over the weirs was then 75.0 cu. ft. per min. and the total of the caisson leakage (clear water entering the dock from various sources), and non-silt-carrying side-wall leakage, 74.6 cu. ft. per min. The flow from the dock floor, therefore, was negligible. There was still in all probability a considerable cavity under the dock, however, and repairs to the dock could not be considered complete or satisfactory until it had been filled.

Grouting Cavity Under Dock

Preliminaries.—Four 2½-in. pipes were suspended from a cable slung across the dock so that the pipes ran straight from the west coping to the relieving

vent. Three of these pipes were connected to the relieving vent with pieces of 2½-in. fire hose, as shown on Figs. 5 and 7, while the fourth was run to an independent grout hole drilled near-by. A 1-in. pipe was slung with the others and connected to the relieving vent to permit the escape of any air that might be carried down with the grout. Before the dock was flooded a light line was made fast to the body of the valve on the relieving vent to enable a diver to find it readily, as the water of the Delaware River is so dirty it is impossible to see anything at a depth of more than 8 or 10 ft. In the meantime, the grouting machines were given a thorough overhauling and set up on the pavement to the west of the dock so that the flexible hose from the battery of machines could be readily connected to any of the four grout pipes. Materials for several hundred cubic yards of grout were also assembled at the machines and arrangements made for the quick delivery of more sand and cement if needed.

Operation.—Grouting of the cavity under the floor was started on Monday morning, December 8, 1921, three complete shifts of men having been organized and trained in the operation of the machines so that there would be no interruptions in flow. The work lasted 112 hours during which time 255 cu. yd. of grout were forced under the dock floor.

The long pipes proved liable to stoppage in spite of the straight leads that had been provided; two of the four pipes choked up prematurely and became useless. All four pipes were thus affected off and on during the progress of the grouting. When the pipe in use choked up, the battery of machines was at once connected to another pipe and efforts were then made to free the disabled one while the grouting continued. A ⅜-in. pipe, long enough to reach down into a grout pipe to its lower hose connection, proved to be most effective in freeing the choked pipes. An air hose was connected to this ⅜-in. pipe so that everything in the grout pipe above the stoppage would be brought out by air-lift action. The pressure of water under the dock would then usually blow out the stoppage and free the pipe.

Neat cement was used for the first few hours of grouting in order to reach the more remote parts of the cavity and provide a good bond between the bottom of the dock and the 1 : 1 grout that was used during the remainder of the operation. As the work progressed frequent inspections were made by a diver of all connections and also for a possible filling of the drainage culverts with grout, by soundings into the culverts from the floor drains, using a long stick. Although all possible care was taken to prevent the entrainment of air, there was undoubtedly a great deal carried below the floor. The 1-in. vent pipe was used to get rid of some of this air and some found its way through small leaks in the floor, causing violent bubbling.

After 108 hours of work, small flows of grout appeared from construction joints in the side-walls of the dock above the water line, indicating that the cavity in the red clay was completely filled and the grout was finding new channels of escape. Grouting was continued, therefore, to the end of the shift "for good measure" and was stopped at 4 P. M., on Friday, December 12, 1921, just 112 hours after it was commenced. The grout was allowed ten days in

which to set, and the dock was then unwatered. Inspection showed that all silt-carrying leakage had been stopped, but that, as had been expected, the clear side-wall leaks had not been affected. No appreciable quantity of grout had found its way into the drainage culverts or on to the dock floor.

Repairing Side-Wall Leaks

The clear leaks through the side-walls and also those into the drainage culverts were successfully caulked with lead wool and oakum after the completion of the grouting operation. This method was also applied to leaks originating in the filling culverts. It then developed that one of the sewers in the dock floor had been imperfectly filled with grout, because of its large content of mud. When this mud was blown out, silt-carrying leaks developed in several construction joints in the drainage culverts, but stopped when the sewer was grouted under pressure.

Results of Erosion Discovered Later

As an aftermath to this repair work it is interesting to note that the erosion of material from the leakage evidently did cause some settlement of the earth alongside the dock, or rather a settlement of the red clay blanket in the vicinity of the fissure in the clay. This settlement made itself known by trouble with the 8-in. terra-cotta sewer from the pump-well toilet, which runs along the lower part of the concrete support for the rail of the dry dock crane. For a distance of 57 ft. on the east side of the dock near the pump-well, the pile foundations and with them the concrete foundation for this crane rail had settled, leaving the upper 10½ ft. of concrete as an unreinforced beam or flat arch carrying the crane. The maximum settlement was 7 in. The dry dock cranes had been operated over this section of their travel, but had not cracked the concrete where the settlement took place. The crack under the crane rail foundation was 2⅜ in. wider on the side away from the dock than on the near side, indicating a considerable lateral movement of the clay blanket toward the dock in the general locality of the worst leaks. The settlement had apparently taken place before the leakage into the dock was stopped as a careful examination of the crane runways showed that they had not settled anywhere except near the old leaks and that the settlement was not progressing. The crane rail foundation was repaired by tamping the settlement crack full of cement mortar or thick grout.

Final Result

Repair work of this kind cannot be pronounced unreservedly a success until the lapse of a number of years without a recurrence of the trouble which the repair attempted to correct. The big dry dock at League Island is accordingly being kept under close observation; in the meantime, it is being used and is giving no indication that further settlement or leakage will occur.

DISCUSSION

F. R. HARRIS,* M. AM. SOC. C. E. (by letter).—The author has so completely described the subject-matter of his paper that little may be said in the way of possible additional information as to the conditions or operations involved in the repair work on the dry dock at the Philadelphia Navy Yard.

As a matter of possible interest to the Engineering Profession, it might be well to review some of the conditions existing before the investigation was made and the repair work undertaken. The writer, who was in charge of the original design and of construction at the time of completion of the structure, has always felt that in a masonry structure of this character, subject to so large a hydrostatic head of water, it would be practically impossible to avoid leakage or at least extensive seepage. In his experience with similar structures he has never found one that did not show some leakage. This leakage usually occurs in concrete and other masonry work, through expansion joints especially provided for, and through shrinkage or settlement cracks, occurring usually during construction or being incident thereto. He has tried to correct this sort of fault, if so it can be called, by special copper or lead strips at such expansion joints or by lead-wool caulking or other type of waterproofing, but in no case with complete success. Such leakage, while perhaps harmful to the structure, is, nevertheless, only so incipiently, through the effect of salt or brackish water on the concrete, resulting in some mechanical or chemical deterioration, so localized as to be readily repaired whenever necessary. This kind of leakage, occurring in various parts of so large a structure as a dry dock, does not seriously threaten the structure in its entirety, especially in the way of a collapse.

The writer has concluded that the flow incident to such leakage would involve movements of water at such low velocities through the subsoil foundation as to have practically no erosive effects. When this is not the case the effect of leakage is usually local and of short duration, so that such small quantities of subsoil would be removed by the water flow as to have practically no effect on the stability of the structure. This statement, of course, does not apply to other types of dry docks, as, for instance, the timber type (formerly called the Simpson dock) much used about twenty or thirty years ago for localities where leakage was likely to cause serious erosion, or undermined and localized settlements, or, eventually, if not checked, collapse of the entire work.

The writer was not unduly alarmed, therefore, when the leakage in the new dry dock at the Philadelphia Navy Yard was first observed, as he felt confident that equilibrium would soon be established. With this conclusion in view, the investigation was ordered more as a precautionary measure than otherwise, the soil content of the leakage being observed in the belief that it would diminish and eventually disappear. This, however, did not occur and when approximate estimates were made indicating that a large quantity of the subsoil beneath the dry dock had been eroded and was continuing to be

* Rear-Admiral, C. E. C., U. S. N.; Public Works Officer, Third Naval Dist., New York, N. Y.

removed, the investigation took a more definite form. It soon became apparent that the voids under the structure approximated several hundred cubic yards and that if this condition were permitted to continue, the rate of soil removal (which the investigation showed was not less than 1 cu. yd. per day and as high as 2 cu. yd. per day), would result not only in serious settlements, but possibly in the destruction of the entire structure costing more than $5 000 000.

The writer has his own theory as to the reason for the continued progressive erosion. The leakage through the masonry was well distributed, in no case being alarmingly great. The area of the structure was of such magnitude that, considering the rate of flow, the velocity would have been so small as to prohibit the removal of any quantity of subsoil material on any other theory than that the leakage found its way under the structure, not by general seepage through a porous soil, but by some one or two definite clay fissures in which the flow was at a sufficiently high velocity to erode and carry away the material, and that, in all probability, in some locations at least, the flow did not occur directly beneath the sub-grade concrete but some distance below, resulting in the caving of the soil above the water flow.

The repair measures have apparently been completely successful. The settlement under the crane rail on the east side of the dry dock near the pump-well occurred after Mr. Angas had left Philadelphia and is, as he stated on page 595, undoubtedly a direct result of leakage and the consequent erosion of the subsoil. The corrective measures were taken none too soon as the 7-in. settlement referred to in the paper would indicate.

Walter Ludlow Taylor,* M. Am. Soc. C. E. (by letter).—Since the completion of the grouting there has been practically no movement of this dock. Readings have been taken periodically on marks established for the purpose and no real change of any part of it is apparent. Several large ships have been in the dock, some of them for a considerable length of time. One of them imposed an unusually heavy load over that portion of the floor which suffered most from erosion of the material under it, and at that time the dock was watched with especial care but no change in the dock structure could be observed.

The work described by the author appears to have served its purpose very successfully. There is now no apparent reason for anticipating further trouble, and the dock is believed to be successfully founded.

W. Mack Angas,† Esq. (by letter).—The writer shares the opinion of Admiral Harris that the continued, progressive erosion of subsoil material from beneath the dock was due to the leakage originating in a high velocity flow through one fissure, or, at most, a few fissures, in the red clay stratum on which the dock is founded. That the rate of leakage through construction joints before repairs were affected was greatly influenced by the opening or closing of the relieving vent and was still further affected by the application of a pump to this vent, indicates that the greater part of the leakage originated

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† Lieut., C. E. C., U. S. N., San Diego, Calif.

in the immediate vicinity of the relieving vent. Somewhat extensive channels, however, must have been eroded between the red clay stratum and the bottom of the dock foundation, as the manner in which the grout found its way from one construction joint to another, clearly indicated the existence of such channels.

The settlement of the crane runway foundations on the east side of the dock also indicates that the erosion was localized, so much so as to cause a decided lateral movement of material toward the eroded cavity. In a letter to the writer, Commander Walter H. Allen, C. E. C., U. S. N., M. Am. Soc. C. E., who was Public Works Officer of the Philadelphia Yard when the repairs were made, gives the following information about the condition he found under the crane rail:

"I have not at hand the letter I wrote reporting this condition to the Bureau so that I cannot quote figures exactly. If I recollect rightly, I figured that the toe of the piles supporting the crane track had moved 18 in. toward the dry dock to cause a difference of 2¾ in. in the width of the crack on the near and far sides of the crane foundation. Undoubtedly the piles would bend inward at the toe with the moving of the earth, so that probably the actual moving of the toe of the piles was twice as much as I figured.

"This to me would indicate that at some time there had been a bodily movement of the dirt around these piles, rather than a gradual seepage of small particles from this locality. The ground to the eastward of the crane rail is rather rough. No settlement was ever detected, although one might have been expected. However, a settlement of a few inches might have taken place without being noticed."

The information given by Mr. Taylor as to the heavy loads which the floor of the dock has sustained since grouting was completed, shows that the cavity under the floor was well filled with grout.

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Paper No. 1563

FACTORS IN THE ZONING OF CITIES

A SYMPOSIUM*

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* Presented at the meeting of the City Planning Division, Detroit, Mich., October 24, 1924.

ZONING AND HEALTH

By George C. Whipple,* M. Am. Soc. C. E.

Introduction

The word "zoning" is now commonly used to designate the governmental process of dividing municipalities into districts and imposing on private property in such districts, uniform building restrictions relating to height, bulk, and use. The New York Committee which set the pace in this matter was called by the descriptive title, "Committee on Building Districts and Restrictions", but the shorter expression seems to be preferred by the American public, even if the districts established bear little resemblance to mathematical zones.

It was natural and fitting that zoning should begin in New York, N. Y., because this largest of cities was suffering most from congestion, indiscriminate building, and the encroachment of private on public interests. The conditions were prejudiciously affecting the welfare of the community as a whole, working damage to property investments and personal injury to individuals. Zoning came to New York as a necessary act of salvation; but it is a measure the need of which has since become recognized in scores of American cities, witnessed by the two hundred and more zoning ordinances which have been passed during the last ten years. So general has zoning become that one may wonder why it is necessary at this late date to consider the fundamental reasons for it or seek to justify its constitutionality. It is not realized, perhaps, that Court decisions resulting from mandamus proceedings and involving the question of constitutionality are relatively few in number and not altogether in agreement. The legal status of zoning cannot yet be regarded as fixed, although each year appears to strengthen its standing. It is worth while, therefore, to consider some of the basic ideas which underlie the restriction of buildings by districts and without which zoning laws would be unsound.

In the typical zoning ordinance, provision is made for several height districts in each of which a limit is placed on the height to which buildings, or parts thereof, may be carried on established street lines, with provision for greater heights for parts of buildings back of certain established planes. From two to five or more height districts are commonly provided, with uniform regulation for each district, the street width being often used as the basis of the mathematical statement of height permitted. Bulk or area districts are covered by regulations which fix the percentage of the lot that may be occupied and the location and extent of open spaces in front of and around the building. Use districts differ according to the permitted or prohibited use of buildings or land, and are described by the words, "residential", "business", or "industrial". The various districts vary greatly in size and shape, and the three kinds of districts are rarely co-extensive; hence, in any zoned city, there are many combinations. As the movement progresses, there seems to be a

* Cons. Engr.; Prof. of Sanitary Eng., Harvard Univ., Cambridge, Mass. Professor Whipple died on November 27, 1924.

tendency to simplify the law by making residence, business, and industrial areas the primary basis of classification, and subdividing each of these on the basis of height and bulk.

It is important that there be uniformity of regulation for each district, for without this the principle of equality before the law would be violated. It is important, also, that the zoning plan be a comprehensive one for the entire city.

Zoning is an essential part of city planning. Generally speaking, about three-fourths of the land area of a city is privately owned and subdivided into blocks and lots; the other fourth, devoted to streets, parks, etc., is owned by the municipality or dedicated to public uses. Again, speaking in generalities, municipal control of the public land is obtained through the governmental power of eminent domain, while municipal control over the use of private property is dependent on the exercise of police power. With rare exceptions, eminent domain has nothing to do with zoning; there is no question of compensation to the owner; no question of the necessity of acquiring private property for public use. The constitutionality of zoning depends on whether the restrictions proposed are justifiable under a reasonable use of the police power, a common law principle which, although undefined and undefinable, finds its backing in certain well recognized needs of the community. Used conservatively, the police power has to do only with injury to health, safety, or morals; used more liberally, it covers, in addition, such matters as the public order and convenience and even extends to what are called the amenities of life. With the increasing concentration of people in cities, there is good reason for the widening scope of the police power which has been witnessed during recent years.

Zoning is advantageous to a city in many ways. It tends to stabilize real estate values, to promote orderly building, to enhance beauty, and to develop local self-consciousness and civic responsibility on the part of the people. Yet, in the face of these benefits, zoning is likely to be declared by the highest Courts to be unconstitutional if it cannot be justified under the police power; and although instances may be cited where the police power has been exercised in a constructive manner to promote the general welfare of a community, its preponderant use has been to prevent injury to health, safety, morals, and—the lawyers like to add—"the like". The purpose of this paper is to outline the scientific evidence bearing on the relation of zoning to health.

Health.—At the outset, it is important to grasp the full meaning of the word, "health", to realize that it is more than the absence of disease; that it has a positive quality; and that it has to do with the mind as well as the physical body. It is useful to keep in mind the derivation of the word from the Anglo-Saxon "haelth", which implied wholeness. If one were to venture a definition, it might be said that health is "that state or quality of life in which the body is sound, the various organs function naturally, and the whole organism responds adequately to its environment."

In a popular sense, public health means the general or collective health of the community. In an administrative or legal sense, it means the health of the community as influenced by factors which affect a considerable number of people in some connected way. The police power is not limited to public health

used in this restricted sense, but deals with health. Attention should be called to the fact that the adjective public restricts the word health instead of amplifying it.

Although it is difficult to define normal health, it is recognized that some factors tend to injure it, or lower its state, whereas other factors tend to promote it, or raise its state above the normal. Normal health presupposes a normal environment, the two ideas being complementary and inseparable. It is the purpose of zoning, as it is that of sanitation, to secure and maintain an environment in which normal human beings can lead normally healthful lives.

In an address* on "Sanitation—Its Relation to Health and Life", before the Sanitary Engineering Division of the Society, the writer pointed out that the principal injurious factors to health are infections, poisons, and accidents. The physiological factors, air, food, water, light, temperature and humidity, sleep, exercise, clothing, and shelter, and the sensatory factors, smell, taste, sound, sight, and touch, are either health-promotive or health-injurious, according to their nature. This classification, indefinite though it is, serves to steady one's ideas when considering the complicated relations between health and environment.

Quantitatively, health can be measured only imperfectly and in part. Individual health may be expressed in terms of growth, height, weight, and other biometrical units. Community health on its negative side may be measured in terms of death rates and sickness rates, general and specific, for different classes, age groups, and particular diseases. No adequate methods of measuring community health, on its positive side, have yet been developed; perhaps they will come in time.

To a large extent, therefore, the subject under discussion is beyond the range of statistics; and reliance must be placed on accumulated experience and the opinions of competent authorities, rather than on logical scientific demonstration, although, in certain parts of the problem, scientific proof is available.

Indoor and Outdoor Conditions.—The relation between health and indoor life has long been recognized. Laws and ordinances covering the size and ventilation of sleeping rooms, drainage, dark hallways, cellars, windows, refuse disposal, and many other items, are common. Detailed building and plumbing codes, housing laws, tenement house laws, and the like are in force in most cities. It is well recognized by the Courts that insanitary indoor conditions are prejudicial to the health of the people. It is coming to be recognized that, in important ways, indoor conditions are dependent on and controlled by outdoor environment. The light that enters a room through a window depends on the light that falls on the outer wall of the building, and this is affected by the position, height, and bulk of neighboring buildings. The quantity of air that enters a building is influenced, sometimes very greatly, by neighboring buildings, and the quality of the air is affected by what is going on in the neighborhood. In fact, nearly all the physiological and sensatory factors related to health may be used to illustrate the close connection between indoors and outdoors.

* See p. 87.

Placing restrictions on the height and bulk of buildings is virtually public control of the space outside the buildings. It prevents private owners from monopolizing light and air to which all people should have a common right. In some respects, time-honored conceptions in regard to property rights are faulty. It is assumed that lots of land privately owned are bounded by vertical planes which extend upward and downward without limit, unmindful of the fact that, in this latitude at least, the sun's rays fall slantingly on the earth and the winds blow horizontally. Building without limit on one's land, therefore, may interfere with a neighbor's use of his land and the enjoyment of certain bounties of Nature, thereby doing injury to his health and comfort. From this point of view, restriction on height and bulk appears to be justifiable.

Conversely, the indoor use of property may affect outdoor conditions. Buildings of great height and bulk lead to such indoor massings of people that not only are the means of ingress and egress provided with difficulty, but means of conveyance and the streets themselves become so congested that safety, health, and morals are jeopardized. Congestion may extend even to the substructures of the streets—the water mains, sewers, gas pipes, and electric light and telephone wires. Municipal governments, responsible for the streets and their use, cannot adequately perform their duties in the face of excessive developments of private property abutting on the streets. The indoor use of property, whether for residential, business, or industrial purposes, controls the character of the vehicular traffic and the character of the pavements required for it; it affects the cleanliness of the streets, as well as dust, odors, sights, and noises. The abutters and the public have common interests in the streets and public lands, which can be protected only by placing restrictions on the use of private property.

Phases of Life.—One of the primary purposes of zoning is to safeguard the conditions which affect three primary phases of life, namely, work, recreation, and sleep, each of which occupies about one-third of the adult's normal day. Adequate provision for work, sleep, and recreation (using this word in a sense broad enough to include rest and nourishment and not merely as a synonym for pleasure), is essential to health. The necessary conditions are not the same for all three, although for sleep and recreation they are not dissimilar. The keynote of work is efficiency; of sleep, quiet; of recreation, cheerfulness.

In infancy and old age, and with the sick of all ages, the conditions which favor sleep are especially important. During childhood and youth, when bodies are growing and minds are developing, the recreation phase controls. In middle age, the work phase predominates. To a large extent, the three phases of life are controlled by the sun—the day is for work, the night for sleep, and the morning and evening for recreation; but to an increasing extent, life in cities ignores the clock. Factories run continuously, night work is required in many ways, transportation never ceases. Those who work at night must sleep by day. What was once a "time" separation is fast becoming a "place" separation. To obtain normal, healthful conditions in cities, home life must be separated in place from work life, and, in order that permanency be given to this separation, a certain amount of governmental control of private property is essen-

tial. This is the basic principle which underlies building restriction by districts.

In making this place separation, it is necessary to take into account various practical considerations. Many people like to live within walking distance of their work, and the daily walk, if not too long, is one of the positive factors of health. Home life requires that the grocer, the butcher, the baker, and other neighborhood conveniences be not too far away. Certain associated businesses gain in efficiency by segregation. Some kinds of manufacturing involve processes which are noisy or which give rise to odors, bearable during work, but offensive from the standpoint of home life. Cities which have been built under the doctrine of *laissez faire,* cannot be rebuilt in a day. These and similar facts have led to the establishment of zones of the most irregular shape, size, and position, zones not always topographically logical, but the best that can be established under the circumstances. The need of zoning is the best argument in favor of city and regional planning.

The primary object of zoning, therefore, is (1) to protect the basic phases of life against injury by providing adequate place-separation of residence, business, and industry; and (2) to prevent the private monopoly of natural light and air, necessary to health, by restricting the height and bulk of buildings in ways appropriate to their neighborhood.

With these general principles in mind, various factors involved in the problem, namely, light, air, noise, odors, congestion, and the like, will now be discussed in some detail.

Light.—The rays of the sun bring light and heat to the earth and both are necessary to man's existence. Dr. Haven Emerson, paraphrasing Michelet, has tersely epitomized human experience by saying, "You cannot raise babies any more without light and air than you can raise plants." Although admittedly mysterious in its action, sunlight is of positive biological benefit and this is true even of diffused sunlight, or daylight. Its action is both physiological and psychological. It is a natural stimulant to the skin and the nervous system. It aids naturally in providing resistance to the body against diseases like tuberculosis. It has recently been learned that it plays an important part in the cure and prevention of rickets in children. It helps to cure tuberculosis of the bones. It provides illumination, the absence of which hampers activities of mind and body and induces eye-strain with its attendant damages and discomforts. It provides warmth in winter. Although science has not yet fathomed the influence of the sun's rays (and this influence may perhaps include the rays beyond those of the spectrum of light), it is a matter of accumulated experience that sunlit rooms are not only cheerful, but healthful, and that dark rooms are gloomy and unhealthful.

There are likewise many indirect benefits. Sunlight is a powerful disinfectant, rapidly destroying bacteria exposed to it, whether floating in the air or resting on pavements, floors, or walls. Unequal heating of the air induces convection currents and beneficial air movements. Places not exposed to sunlight are more likely than others to contain stagnant air. Air movements have an important influence in regulating the temperature of the body. Stagnant

air around the body tends to increase in humidity, thereby making a person feel warmer in summer because of lessened evaporation and cooler in winter because of greater conduction of heat by the moist air.

Sunlight tends to reduce the relative humidity of the air by increasing its temperature and its ability to hold water vapor. By removing moisture from dust particles in the air, it tends to lessen fogs. It also tends to dry pools of water which otherwise might become breeding spots for mosquitoes.

Sunlight markedly influences vegetation. Trees, shrubs, and grass are natural automatic regulators of heat conditions. During the summer, trees produce desirable shade, yet, in winter, they do not obstruct the sunlight. In this respect, the shade of trees differs from the shade of buildings. Vegetation also provides a natural chemical balance. Human beings, as well as all animals, inhale oxygen and exhale carbonic acid; whereas plants in sunlight take in carbonic acid and give out oxygen. Vegetation cannot thrive without sunlight and water. It is a matter of history that the increasing height of buildings and the increasing extent of impermeable area due to buildings and pavements, drive out trees, shrubs, and grass. The effect of vegetation is local. Trees and grass concentrated in parks cannot take the place of vegetation on streets and individual house lots.

Daylight, which means indirect lighting from the sun by reflection from the sky, the clouds, and various surfaces, does all these things, but to a less degree than direct sunlight. Sunlight may even be too great, as every one knows, especially during the summer and in the tropics. Daylight has an important economic value. It is not only beneficial physiologically and psychologically, but increases the productiveness of labor and reduces the necessity of artificial illumination. Artificial illumination involves expense and must be arranged with great care in order to be effective and not cause injury to the eyesight. Lighting with oil or gas tends to vitiate the air by increasing the carbonic acid and moisture, and even by increasing the poisonous carbonic oxide.

Artificial lighting also increases fire risks. Lack of proper exterior lighting increases the window space required and this, in turn, increases the heat loss in buildings in winter.

There are abundant reasons, therefore, for stating that adequate provision for allowing daylight to enter an inhabited building is essential to human growth, health, vitality, and comfort. Whoever, by building overmuch on his own land, prevents his neighbor from receiving a reasonable amount of light on his land is doing him an injury that properly comes within the scope of the police power.

Much can be done to make the best use of sunlight by the orientation of buildings and streets. Buildings facing the cardinal points are not as well lighted throughout the year as those facing the quarter-points. Western townships with their north, south, east, and west boundaries have tended to grow up into cities having streets in these directions. Many trivial matters often control street orientation, whereas the element of sunlight receives scant attention. The matter does not become one of real importance until high

buildings are constructed, and, by that time, street lines have become fixed. Contact with civil engineering students in recent years has convinced the writer that astronomy receives too little attention in the schools. Few students, on graduation, are able to trace the sun's path in the heavens at different seasons or to draw the shadow of an isolated house, not to mention the shadows of high buildings on each other, when located on a street of given latitude, width, and direction.

Air.—The necessity of pure air need not be argued. It is a fundamental principle registered by human experience. Modern studies of ventilation emphasize the physical properties of the air—temperature, humidity, and movement—and their physiological importance. These heat relations are closely linked with the problem of sunlight, already considered.

Nothing in recent experimentation, however, controverts the need of cleanliness of the air we breathe. Dust in the air tends to irritate and clog the breathing apparatus. If the dust particles are sharp, as in the case of silica, they wound the delicate membranes so that bacterial infection is likely to follow. Statistics of tuberculosis among stone-cutters show that this disease is prevalent in direct proportion to the percentage of silica in the stone dust. Dust may injure the eyes and clog the pores of the skin. Its damage is economic as well as physiological. The extent to which disease germs are transmitted from person to person through the air is not well known. Ordinarily, spray from the mouth or nose does not carry more than a few feet, and accompanying bacteria capable of detection by present methods do not live long in the air because of the destructive effect of drying and sunlight. The behavior of the filterable viruses in air and the longevity of the spores of bacteria, moulds, and fungi, however, are only imperfectly understood. Irritating fumes from chemical processes may be not only offensive to the senses, but also cause physical injury. Any air which by reason of dust or bacteria, irritating fumes, or offensive odors, tends instinctively to induce shallow breathing, must be regarded as injurious to health. Just as pure air tends to promote health by naturally inducing deep breathing and stimulating the bodily functions, so exposure to vitiated air tends to break down the individual's power to resist disease, especially respiratory affections, such as colds, pneumonia, and tuberculosis. Here, the element of time is important. A fleeting bad odor may be offensive, but do little or no injury, whereas some odors, long continued, may be injurious. On the other hand, there are odors to which people become accustomed and which do no damage. Individual susceptibility plays an important part in the phenomenon of odor. The extent to which foul air affects breathing during sleep appears to be not well known from experimental studies, but judging from experience, its influence is quite as important as during waking hours.

The air which enters a building, both in quality and quantity, is influenced by the neighboring buildings and by the streets. Intakes of ventilation systems are more often located with reference to indoor distribution than to exterior conditions which affect the quality of the entering air.

Many studies have been made of the number of dust particles and bacteria in city air, both in the United States and abroad. The absolute figures need not be considered because their order of magnitude varies according to the methods used and the sizes of the dust particles included in the counts. Relatively, the tests agree in showing that dust in the air is greatest near the street and decreases logarithmically upward; that macadamized streets and much traveled granite pavements produce more dust than sheet-paved streets; that dust is closely associated with the cleanliness of the streets and methods of cleaning; that automobile traffic produces less dust than horse traffic, but distributes it to a greater extent; that street cars raise dust one or more stories higher than horse traffic; that less dust is found over grass land than pavements; that less dust is found in residential districts than in business or industrial districts, etc.

Smoke is another important source of dust. The use of oil-burners in place of coal-burners is changing this problem. The Mexican oils are higher in sulphur than American oils, and their use increases the sulphurous fumes in the air to a measurable extent.

Where high buildings exist, the ventilation of streets is coming to be an important problem. If buildings are high relative to the street width, there is likely to be a stagnation of air over the pavement and a concentration of dust, bacteria, foul odors, and automobile smoke, injurious to the health of persons using the streets.

The density of automobile traffic in cities is already so great that traffic officers are sometimes overcome by the poisonous fumes of carbonic oxide, and pedestrians are greatly inconvenienced by the smoke. In business districts, where large trucks are used and traffic is heavy, these conditions are especially bad, and are at their worst when associated with high buildings with flat roofs and overhanging cornices. If the streets have a marked grade, there is a tendency for gravity currents to produce partial ventilation with dilution of the bad air; but when they are level, gentle winds do not suffice to effect the necessary ventilation of deep, cavernous streets. Strong winds, on the other hand, produce excessive currents through cavernous streets, that are very objectionable in winter.

In the interest of air purity, therefore, zoning is justified. Residential districts, where people sleep and recreate and where children grow up, need protection against the atmospheric dirt of the business and industrial districts.

Noise.—Susceptibility to noise in general and to particular noises varies greatly among individuals. It is a difficult question to discuss. It is well-known that noises hinder sleep. Physicians say that certain persons, especially those suffering from nervous diseases, are seriously injured by noise and vibration. Every one knows that in many ways noises interfere with the comfort and tranquillity of life. Quiet is especially important at night, in residential districts, and near hospitals and schools.

Noises are greatly increased by the reflection of sound waves from the hard surfaces of pavements and building walls. Limitation of the height of buildings is, therefore, a means of noise reduction. Vegetation, on the other

hand, tends to dampen sound waves—another reason for providing conditions favorable for trees and grass in residential districts.

Many kinds of noises are preventable, but others appear to be inseparable from traffic, business, and manufacturing processes. In these cases, segregation appears to be the best solution.

Congestion.—Congestion, or crowding, needs to be viewed from at least three angles as far as health is concerned, that is, room crowding, land crowding, and personal contact.

Room crowding is commonly expressed as a ratio of the number of square feet of floor area, or number of cubic feet of room volume, per person. Minimum limits are sometimes placed on one or both of these ratios for sleeping rooms, barracks, schools, factories, etc., based on the hygienic need for light, air, and ventilation—matters which have already been considered.

Land crowding, expressed as so many persons per acre, introduces two additional elements, the number of stories and the area of the building with reference to the size of the lot and the street width. One of the most important reasons for restricting the height and bulk of buildings by districts is to prevent overcrowding of corridors, elevators, streets, and sidewalks. These have to do more with questions of safety and accident than with normal health, questions not considered in this paper.

The third phase of congestion bears directly on the spread of disease. When people are brought into such close contact that opportunity exists for breaths to intermingle, as in crowded elevators and cars, or for the nasal spray of one person to pollute the air breathed by another, there is serious danger that disease germs may spread and that colds and respiratory diseases may become epidemic. It may be true, as medical bacteriologists claim, that crowd exposure tends to build up an acquired immunity against certain diseases so that to some extent Nature protects itself, but the fact remains that, on the whole, crowding speeds up and increases the transmission of disease. It is a menace to health, morals, and safety.

No one has yet established a logical basis of street capacity, either for pedestrians or vehicular traffic, or the relation which an adequate street capacity should bear to the size of abutting buildings. Most streets in American cities were laid out to accommodate slow-moving traffic and buildings of 2, 4, or 6 stories, or thereabouts. Increase in building height has led to serious street congestion in many places. Fragmentary data exist as to the number of square feet per person in buildings used for different purposes, the permissible capacity of elevators, the space occupied by moving pedestrians under different conditions, and the street space monopolized by vehicles of different character moving at different speeds. These data should be assembled and studied with a view to establishing, if possible, some reasonable relation between building size and street area. The writer's unsatisfactory attempt to do this (too meager to warrant publication), has convinced him that the fundamental data need first consideration.

Psychological.—Health is mental as well as physical. Mental health is intangible, but none the less real. Sunlight is beneficial largely because it is cheerful. Trees and grass and flowers are healthful for the same reason. The

beauty of form, color, light, and shade conduce to mental health. Eyes are rested by a change of focus and ears by a change of sound. Monotony causes mental fatigue, and carried to the yielding point, may cause insanity. Children, especially, need opportunities for proper development and adjustment of the senses, but all workers like to get away from their work at night. A most important benefit of zoning is to provide opportunities for the changes necessary to normal mental health.

Community Health.—It is easy to object to particular applications of the zoning principle. Building restrictions of necessity must be arbitrary. Boundaries of districts must be actual lines, and in establishing lines where conditions grade almost insensibly one into another, it is difficult to avoid individual injustices. It is often difficult to show that zoning prevents injury to the health of certain particular individuals. There are various matters for adjustment and administration which should be provided for, as well as may be, in zoning laws. Although zoning as a principle has abundant justification under the police power, it must not be forgotten that since Magna Charta, the individual has had protection against undue restrictions of government in what is known as "due process of law".

The relation between zoning and health is a mass relation. It is the health of the community, the collective health of many people, that is at stake. Families rightly separate working quarters from sleeping quarters; cooking and eating and sleeping in the same room is regarded as insanitary. Tenement-house laws, factory regulations, building codes, and the like, safeguard the internal uses of the buildings. The zoning law does for a city what some of these laws do for the factories, schoolhouses, and dwellings.

When cities grow without plan, their constituent districts tend to change in character. Single houses give way to apartment houses; residential districts are insidiously invaded by business and manufacturing; and old buildings are converted to uses for which they were not intended and for which they are ill adapted. Converted houses are notoriously likely to be insanitary and unhealthful. In a growing city, there is a natural tendency toward concentration for economic reasons. A person who erects an apartment house in a region where only single dwellings exist, is capitalizing for his own pocketbook, light, air, and the other residential benefits at the expense of his neighbors. A single-house region once infected with an apartment house tends to accumulate other apartments, and the neighborhood tends to change from a stable, house-owning population to a shifting, renting class, a class lacking in neighborliness and civic pride and leading an impoverished family life. Thanks to sanitation and other modern improvements, apartment-house life has been made healthful for adult existence, but the compressed and repressed life of a modern city apartment is not conducive to growth or to a life that is full and rich. Segregation of apartment houses is justified as a measure for protecting community health.

Gradually it is dawning on men's minds that cities which grow to great size do so at the expense of the health and comfort of their own citizens; that rapid growth which outruns municipal ability to make or remake necessary

thoroughfares and provide needed public utilities, leads to ugly confusion, whereas a slower, well-ordered growth is more likely to lead to civic beauty and a better civilization. The United States is entering on a period of lower population increase. As pride in growth and quantity production lessens, as it must, the elements of stability and self-control and beauty need to be strengthened.

Zoning should be regarded as a sort of collective self-control, a means by which a city controls its own life and growth for the best good of all its citizens. It is an act of police power fully justified in the interest of morals, safety, and health.

ZONING FOR USE

By Edwin A. Fisher,* M. Am. Soc. C. E.

Introduction

The comprehensive paper† by Morris Knowles, M. Am. Soc. C. E., on "The Purpose of Zoning Ordinances, and Methods of Administration," and his equally admirable contribution to the subject of zoning presented at the Annual Meeting of the Society, January 19, 1923,‡ cover the general subject of zoning very fully. The speaker, therefore, will confine himself to a description of methods and accomplishments of zoning for use in the City of Rochester, N. Y.

The Enabling Act

In Rochester, zoning for use is done by the City Planning Bureau acting directly by authority of the Legislature, as shown in the following provisions of the City Charter:

"Section 290.—City Planning Bureau.—The City Planning Bureau is in the Department of Engineering, and the chief officer thereof is the Superintendent of City Planning appointed by the City Engineer to hold office during his pleasure. He has power, with the approval of the City Engineer, to appoint to hold office during his pleasure such subordinates as may be prescribed by the Board of Estimate and Apportionment." (C. 505, L. 1917.)

Section 291 defines the powers and duties of the Superintendent of City Planning, as follows:

1.—The Superintendent has power to accept streets.

2.—The Superintendent must approve of all plans for opening, widening, extending, or discontinuing streets, or fixing the widths of pavements or sidewalks.

"3.—It is the duty of the Superintendent of City Planning to prepare a city plan and to set forth thereon streets which the proper development of the city requires to be opened, widened, extended or discontinued, and the width thereof; also the sewer systems necessary to be constructed or extended, and the water mains necessary to be laid or extended, and the location of buildings, docks, parks, playgrounds, school houses and municipal buildings. Such plan shall cover the territory embraced within the boundaries of the City of Rochester and also all territory within one mile of such boundaries, and such further territory outside of the city as the Superintendent may deem proper. Such plan may be made in parts from time to time covering different portions of the city or territory outside of the city. Amendments, alterations and additions to said plan or plans may be made from time to time."

Paragraph 4 of Section 291 refers to zoning for use, and reads:

"4.—The Superintendent of City Planning has power to divide the city into districts and to regulate, restrict and prohibit the location of businesses, trades and industries, and the location or alteration of buildings or struc-

* Cons. Engr. to the City of Rochester, Rochester, N. Y.

† See p. 624.

‡ "Zoning—Its Progress and Application", *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1349.

tures designed for specified uses or of a certain character or class in any such district. A portion of a street may form or be a part of any such district. Residence districts may be created in which shall be permitted only single-family dwelling houses or dwelling houses or residences containing accommodations for such number of families as may be deemed proper. Specified businesses, trades or industries may be permitted in one district, and others excluded therefrom, and special regulations may be prescribed for the business, trades or industries so permitted in such district. Regulations, restrictions and prohibitions in one or more districts may differ from those of other districts. The use or occupation of a building or structure in any district must not be changed so as to be in violation of the regulations, restrictions or prohibitions applying to such district. Such regulations, restrictions and prohibitions shall be designed to promote the health of the public, for the safety and welfare of the inhabitants of the city, for the promotion of the growth and prosperity of the city, and to secure the proper development and upbuilding of the city." (C. 505, L. 1917.)

Section 292 of the charter refers to the City Planning Advisory Board, and reads:

"The City Planning Advisory Board consists of the Corporation Counsel and the four members appointed by the Mayor, and it must annually elect one of its members president thereof, and has power to adopt rules and regulations for the transaction of its business, and to hold public hearings upon matters coming before it, and it must hold a public hearing upon the written request of two members. It is the duty of the Superintendent of City Planning to submit all fixations of the width of pavements and sidewalks and all proposed plans and the proposed formation of zoning districts and regulations in reference thereto, and proposed action with reference to accepting, opening, widening, extending and discontinuing streets, to the City Planning Advisory Board, and upon approval by the Board, the same become effective. In case any such matter so submitted is disapproved by the City Planning Advisory Board, the Superintendent may present the same to the Mayor, and the Mayor, after a public hearing, may approve the same, and it thereupon becomes effective." (C. 524, L. 1921.)

Section 293 provides for the employment of experts:

"Employment of Experts.—The Superintendent of City Planning has the power with the approval of the Mayor to employ experts for specific work or for a specified period of time not extending beyond the expiration of the fiscal year, whose compensation must be fixed by the Board of Estimate and Apportionment, but in each instance the expert so employed must first be approved by the City Planning Advisory Board." (C. 505, L. 1917.)

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"Section 295.—Violations, How Punished.—Any person violating any rules, regulations, restrictions or prohibitions made by the Superintendent of City Planning and duly approved, as provided in this article, shall be guilty of a misdemeanor, and on conviction thereof shall be punishable by a fine not exceeding one hundred fifty dollars, or by imprisonment not exceeding one hundred fifty days, or by both such fine and imprisonment, or by a penalty of five hundred dollars to be recovered by the City of Rochester in a civil action." (C. 524, L. 1921.)

As amended in 1921, Section 126 provides that the City may maintain action in the Monroe County Court, the Supreme Court, and other Courts of record of competent jurisdiction to restrain violations of penal and other ordinances of the Common Council, and of rules, regulations, restrictions,

and prohibitions made by the Superintendent of City Planning and duly approved.

As amended in 1921, Section 202 provides that before the owner of any tract or parcel of land in the City of Rochester sells or conveys any part thereof in subdivisions of such tract or parcel, he must cause a map of such tract to be made, which map must first be approved by the Superintendent of City Planning and by the Advisory Board before it can be filed in the office of the County Clerk, etc.

It will appear, therefore, that the Superintendent of City Planning has greater power, and more extensive duties, than is usual for a City Planning Commission. City Planning Commissions usually report to the Common Council or other legislative body of the city. For the City of Rochester, the Superintendent reports his action to an Advisory Board composed of four citizens, appointed by the Mayor, also the Corporation Counsel, *ex officio*. The action of the Board is final, except that an appeal may be taken by the Superintendent to the Mayor. The Advisory Board, pursuant to the authority granted, has adopted rules of procedure, one of which will permit any property owner aggrieved by the action of the Superintendent to bring the matter directly to the Board for a hearing. This rule was adopted at the request of the Superintendent. Members of the City Art Commission may be appointed to the Advisory Board.

Method of Procedure in Zoning for Use in the City of Rochester

Atlas maps, uncolored, of the entire area of the city were procured, and on these maps the actual use conditions of all the property was shown in colors. After a study of these maps, and numerous hearings before various organizations, the zoning for each particular section of the city was shown on a tracing superimposed on the map and strongly fastened thereto.

This was found to be a very useful method of procedure in designating the various zones. These maps have been preserved and amended from time to time, and are of great use in exhibiting to any property owner the reasons for the zoning as shown on the tracing. They are in constant use in the office of the Superintendent.

The rules and regulations for "use" districts are similar to those in other cities. The "use" districts into which the city is divided consist of six classes, known as Unclassified Districts; Unrestricted Districts; Industrial or Manufacturing Districts; Commercial or Business Districts; Residential Districts Class *D*; and Residential Districts Class *E*.

It will be noted that Residential Districts are divided into two classes; the first class, or Class *E*, provides for single or two-family dwellings, double houses or Boston flats, clubs, churches, convents, schools, libraries, private offices of a physician, surgeon, or dentist, railroad passenger stations, real estate signs referring to the property on which they are maintained, recreational buildings in public parks and playgrounds, farming, truck gardening, nurseries, and greenhouses; also, for purposes necessarily or ordinarily acces-

sory to such usage, as, for example, a private garage for not more than four motor vehicles.

The other class of Residential Districts, referred to as Class *D*, covers, in addition to the uses specified in Class *E*, tenement houses, hotels, lodging houses, boarding houses, etc.

The rules and regulations for "use" districts, together with the maps made a part thereof, were presented by the Superintendent to the Advisory Board and approved by the Board on September 22, 1919. Zoning for use, therefore, has been in force in Rochester for four years.

The speaker acted as Superintendent of City Planning from the inception of the work until January 1, 1924, in addition to his duties as Consulting Engineer, and since January 1 has been more or less familiar with the operations of the Bureau as Consulting Engineer.

In a discussion* on "City Planning" before the Annual Meeting of the Society on January 19, 1923, the speaker gave a brief description of the work in Rochester to that time. He stated as his belief that "zoning that cannot be changed without too much trouble had better not be done".

The rules and regulations of the City Planning Bureau in Rochester provide for changes and exceptions. Among these provisions are the following: The Superintendent of City Planning, with the approval of the Advisory Board, shall have the power to construe these rules and regulations liberally to bring about the greatest public good. From time to time, on his own motion, or on petition, after public notice and hearing, the Superintendent, also, with the approval of the Advisory Board, may amend, supplement, or change the rules and regulations and the districts as established on the "use" maps.

If the owners of 50% or more of the frontage in any street between intersecting streets present a petition to the Superintendent of City Planning requiring a change or repeal of regulations provided for such frontage, the Superintendent must present the petition to the Advisory Board which must act upon it within a period of not more than sixty days.

Exceptions.—The Superintendent, with the approval of the Advisory Board, may, in writing, authorize exceptions to the "use" rules and regulations, when he deems such exceptions to be necessary for the public good:

"The erection in a residential district, for a period of not more than one year from the date of such authorization, of a temporary building for commerce or industry incidental to the residential development.

"The issuance of a temporary and conditional authorization for not more than two years for structures and uses in contravention of the requirements of these Rules and Regulations in undeveloped sections of the city.

"The enlargement of an existing structure or use, or the erection on the same lot of an additional structure for a lawful use existing at the time of the adoption of these Rules and Regulations, not conforming to the Rules and Regulations of the Use District in which it is maintained, when such alteration, extension or addition is determined by the Superintendent of City Planning not to be detrimental to, or tend to change, the character of the neighborhood.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1375.

"The establishing of a public garage, public gasoline station, or both, or a stable for more than three horses, in any class residential district upon presentation of written consent of the owners of not less than 75% of the frontage of all real property within 200 ft. of the lot upon which the said use is proposed, where, in the opinion of the Superintendent and the Advisory Board, such use will not be detrimental to, or tend to change, the character of the neighborhood. On a corner lot the frontage shall be measured on the street to which the use proposed is to have vehicular access. In computing the percentage of consents required under this provision, the frontage of lots having a usage of a public garage, public gasoline station, or a stable for more than three horses, shall be counted as consenting. A map or plan of the entire property drawn to scale upon which said use is requested, showing the location of existing and proposed structures or buildings upon the premises, and the location of driveways and entrances from the public street or alley, must be submitted with each application, which map shall be kept on file in the office of the Superintendent of City Planning. (Amended November 27, 1922.)

"The establishing of a public garage, public gasoline station, or stable for more than three horses, in a Commercial District, after notification to the owners of frontage on both sides of any street to which said garage, station, or stable is to have vehicular access for a distance of 200 ft. of the lot on which said use is proposed, provided that whenever a written protest against such garage, station, or stable, duly signed and acknowledged by the owners of 20% of the frontage herein specified has been presented to the Superintendent of City Planning, the Superintendent will present the same to the Advisory Board, which will act upon said protest within a period of not more than 60 days after the filing of the same by the protestants with the Superintendent of City Planning. (Amended November 27, 1922.)

"The establishing of a telephone exchange or electric sub-station in a Residential District where such use is determined by the Superintendent of City Planning not to be detrimental to, or tend to change the character of, the neighborhood.

"The erection of an accessory structure in a Residential District, or in a Residential District Class *E*, other than as provided in Section 6 or Section 7 of Article III, where in the opinion of the Superintendent and the Advisory Board, such use will not be detrimental to, or tend to change, the character of the neighborhood. (Amended February 13, 1922.)

"The erection and maintenance of commercial and industrial uses other than hereinbefore prescribed in a Residential District, or in a Residential District Class *E*, subject to appropriate conditions and safeguards, and where, in the opinion of the Superintendent and the Advisory Board, such use will not be detrimental to, or tend to change, the character of the neighborhood." (Amended February 13, 1922, after a decision of the Court of Appeals construing the New York City regulations.*)

Acting under the authority granted by the Legislature and the Rules and Regulations of the Bureau, the Bureau has granted exceptions for various purposes. It will be noted that the regulations prohibit gasoline stations and garages in Residential or Commercial Districts, except by consent of the Bureau. From the time the regulations took effect in September, 1919, to January 1, 1924, the Bureau has granted exceptions for 106 gasoline stations, an average of about 26 per year, nearly all in Commercial Districts.

A careful investigation of every application for a gasoline station is made by the Superintendent, and the applicant is required to file sufficient plans to show the general features and appearance of the building. The type of

* People *ex rel.* Sheldon *vs.* The Board of Appeals, etc., 234 N. Y., 484.

construction of buildings used for that purpose is now much better than when the Bureau began its operation.

Any garage having a capacity for more than four cars is called a public garage; therefore, when a property owner desires one or more additional cars he must proceed in the same manner as if he were asking for a large public garage. The total number of exceptions granted for garages to January 1, 1924, is 140; 118 in Commercial Districts and 22 in Residential Districts. Of the latter nearly all were for one or more additional cars.

Among other exceptions may be mentioned tailor shops, in both Residential and Commercial Districts. The manufacture of ready made clothing is a large industry in Rochester. In the Residential District, both the manufacturers and employees desire to use small buildings—not more than two stories in height—for this industry. The Bureau, therefore, at the request of the residents, made exceptions for such purpose. To January 1, 1924, 56 exceptions have been granted for this purpose; 16 in the Commercial Districts and 40 in Residential Districts.

Changes of Zone

Although no general changes in zone have been made, there have been numerous minor changes. These changes were made largely in the part of the city that is changing from Residential to Commercial or Industrial. One reason for many of the changes is due to the fact that, in preparing the original zoning of certain streets which evidently would be entirely business in the near future, the Superintendent subdivided the street allowing those parts occupied by good residences to be classed as Residential, and the remainder as Commercial. In many cases the demand for additional business has exceeded expectations and changes have been necessary. To January 1, 1924, there have been 173 changes in zone—mostly minor changes, and largely from Residential to Commercial.

As stated previously, the City has no separate and distinct Board of Appeals. The Advisory Board acts as such Board and gives careful consideration not only to matters brought before it by the Superintendent, but also to any matter brought before it by any citizen under the provisions of the special rule heretofore referred to.

Use zoning is generally approved by the citizens of Rochester. Its success is due largely to the high character of the members of the Advisory Board.

Court Appeals

In Rochester, only three cases have been appealed to the Courts, and a decision has been reached on one case only. The first case related to an alleged discrimination in denying an application of one person for a gasoline station and granting the application of another in the same district. It had no relation to the general question of zoning.

The second case, now in the Courts, questioned the right of the Bureau to change the zone from Residential to Commercial.

The third, and most important case, was an application of a property owner for a mandamus to compel the Superintendent of Building to grant a permit

for the erection of an apartment house in a part of East Avenue zoned as Residential Class *E*, that is, for one or two-family houses. In this action, the constitutionality of zoning was raised. The Court, in denying the application, said:

"The Court will take judicial notice that East Avenue is the chief residential street of the city. With these assumptions and under the allegations in the petition, the sole question is whether the city may prohibit the erection of apartment houses in the portion of East Avenue in question, not this particular apartment house, but any apartment house. If the statute and rules are unconstitutional, of course no application to the Superintendent of City Planning and Advisory Board was necessary and the petitioner would be correct in his position that the Superintendent of Building could be compelled to grant a permit, but the statute and rules are not unconstitutional. The use of private property is not unrestricted under the Constitution and the Legislature may from time to time add new restrictions which are reasonably necessary. It cannot be said that the present restrictions as to apartment houses are unreasonable or unnecessary or that they are without the purpose of the statute or the powers of the Legislature.

"The provisions of the Constitution as to the taking and use of private property are to be construed in the light of existing social conditions and advancing social necessities. The powers of a municipality under legislative authority to limit and restrict the use of private property has been sustained in many cases and zoning statutes, ordinances, and rules, substantially similar to the ones involved on this application, have been upheld. The papers, therefore, raise no question of fact and upon the facts stated are insufficient as a matter of law."

AREA ZONING

By Harland Bartholomew,* M. Am. Soc. C. E.

The remarkable spread of zoning in American municipalities has been due primarily to the popularity of the use regulations. Many ordinances deal exclusively with use regulations, and a few embrace use and height regulations. Where use regulations are not the sole concern, attention and discussion usually center about height regulations. Area regulations are seldom questions of particular concern or public discussion.

Of the 261 zoning ordinances in effect in American cities, according to the latest bulletins of the U. S. Department of Commerce (August 30, 1924), 138 contain area regulations as part of a comprehensive zoning ordinance. The cities possessing ordinances with area regulations are divided, as follows:

California	6	Nebraska	1
Connecticut	1	New Jersey	42
Delaware	1	New York	24
District of Columbia	1	North Carolina	1
Georgia	1	North Dakota	1
Illinois	20	Ohio	11
Indiana	3	Oklahoma	1
Kansas	2	Pennsylvania	1
Maryland	1	Rhode Island	1
Massachusetts	10	South Carolina	1
Michigan	3	Tennessee	1
Minnesota	2	Washington	1
Missouri	4	Wisconsin	7

The literature of zoning contains very little data on area zoning. Study of a large number of ordinances containing area regulations reveals great lack of uniformity of practice, and one may seek in vain for any important decisions in the higher Courts on the area provisions of zoning ordinances.

Area zoning is an almost untouched field. One of the best informed attorneys engaged in zoning practice is of the opinion that all zoning will succeed or fail in the Courts depending on whether it can be justified as an exercise of the police power in the interests of public health. This implies that area regulations in zoning ordinances are of equal, if not of greater, importance than the use or height regulations. Considered in conjunction with use or height regulations, area zoning is not of secondary importance.

A brief study of the application of area zoning in cities leads one into the whole field of land economics and housing which to-day are world problems. Are these not worthy of the consideration of the engineer? Is any one better equipped to solve and interpret these problems? The improper adjustment of dwellings to land in cities produces social and economic problems

* Engr., City Plan Comm., St. Louis, Mo.

of great magnitude and is the primary cause of much of the wasteful and unscientific methods of city building.

An examination of the maximum, minimum, and average density of population per unit of land area in a considerable number of large cities is sufficient evidence that overcrowding of land is not an economic necessity. Various forms of housing accommodations (that is, tenements and residences) are needed for certain proportions of the population in any city. The present wide diversity of accommodations and types of dwelling construction may be attributed largely to the lack of a proper understanding of desirable standards that might be produced by proper area zoning as a part of a comprehensive zoning and city planning program. It must be frankly admitted that there are no proper standards generally accepted as such to-day. Although much intelligent and sustained effort has been expended in the preparation of housing laws, it must be admitted that what might be termed satisfactory housing laws have been enacted in only five or six States and, even in these, overcrowding of the land has proceeded apace. Examination discloses no well-defined lines of demarcation between housing laws, tenement house codes, building codes, and zoning ordinances in various cities and States.

In the Central-Western and Southern States, it is not uncommon to find cities in excess of 100 000 population without housing laws, building code, or zoning ordinance. In cities of less than 50 000 or 75 000 population, it is not uncommon to find the most lackadaisical enforcement of any housing or building code regulations that may exist. Growing cities usually witness a gradual decrease of home ownership and lowered housing standards. A vast proportion of the population in large cities (those who through force of necessity or otherwise are "renters") never appreciates nor is in a position to demand good housing accommodations. Land subdividers and builders are more often concerned with maximum economic returns than with the provision of desirable housing standards. The speculative apartment-house building that has accompanied the post-war period well illustrates this fact. In one large city, apartment houses built in 1921 provided approximately 625 sq. ft. of lot area per family (ranging from 400 to 1 200 sq. ft. of lot area per family). In this same city, in 1922, of all the apartments constructed only two provided densities of less than 200 sq. ft. of lot area per family housed. In 1923, in the same city, the tide turned backward and of all the apartments constructed only two structures were built which provided less than 400 sq. ft. of lot area per family. This is a typical example of the fact that good housing standards are entirely secondary to maximum economic returns on anything for which the public can be made to stand. In this particular case, the public was squeezed on the land until it rebelled and began to demand and secure larger accommodations.

In attempting area zoning the first inquiry is whether present practices with their constantly changing vagaries shall be accepted, or whether preferably an attempt should not be made to establish proper standards and limitations. This is no easy task. It involves conflict with structures previously constructed without proper regulation, and conflict with the land and building

speculator who invariably resents any attempt at regulation. It also implies proper adjustment between housing laws, building codes, and zoning ordinances. The city planner cannot ignore the inter-relationship of these three important means of creating desirable dwelling construction. Each in itself is sufficient to meet completely the problems with which the city planner wishes to deal. They must be simultaneously considered and co-ordinated.

As a basis for discussion it is suggested, therefore, that each State enact a proper housing law and that each city adopt a proper building code and zoning ordinance. It is further suggested that the housing law concern itself with questions pertaining to sanitation and proper interior design; that the building code provide for proper forms of construction in the interest of public safety; and that the zoning ordinance provide for the placement of buildings on lots, the regulation of light and air through open spaces within and about buildings, the regulation of building heights, and the segregation of buildings according to types of use.

If the foregoing suggestions can be accepted, the next consideration should be the provisions that should be included in the zoning ordinance in zoning for area.

Front yards (sometimes called "building lines" or "set-backs"), side yards, rear yards, and courts constitute the open spaces about buildings and would appear to be legitimate forms of zoning for area. Their exact determination, however, is somewhat dependent on the homogeneity of districts which implies co-ordination of use and height regulations. The regulation of percentage of lot occupancy by buildings and control of density of population may also conceivably influence the design of open spaces and, therefore, are properly elements of zoning for area.

Before further discussing the exact nature of these forms of regulation the speaker will mention the matter of land subdivision. Probably the most prolific fundamental cause of bad housing, as well as of other evils, is the widespread method of subdividing land into streets and lots of more or less uniform or standardized sizes, regardless of topographic conditions, position in the city structure, and probable use. The exact nature of land use and its early development would seem to be a prerequisite of land subdivision. More and more realtors are coming to realize the importance of this self-evident fact. Land subdivision rules which seek to create a proper subdivision of land in accordance with its ultimate use are being adopted by the city planning authorities. The realtor who subdivides properly, who develops and restricts his property with due regard to its position in the city structure, has accomplished all and more than the city planner. It is failure to complete this full operation that leads to conflict and misunderstanding, to wasteful and inappropriate development of property.

To interject further, it must be recognized that the modern city is so vast and intricate an organic structure that to function with any degree of efficiency and satisfaction it must have unity of plan. Land subdivision and zoning are elements of the comprehensive city plan. They cannot be considered alone. If this is a curb on individual liberty or enterprise, its com-

pensation is greater public and community benefit and convenience. In practice, it still permits of unlimited initiative and enterprise in the development of private property with reasonable, if not unlimited, economic return. City planning and zoning direct rather than restrict the use and enjoyment of private property, and the total cumulative result is greater, rather than less, economic return, not to mention the larger compensations of improved living and working conditions.

Zoning for area, therefore, necessarily implies consideration of the entire city structure and its plan. It implies determination of standards compatible with good housing. Styles of dwellings vary in different cities and even within each city. Some considerations are offered as follows.

Front Yards.—Front yards should not be required in commercial or industrial areas under zoning ordinances. If the streets are too narrow to admit adequate light and air, the building height should be restricted in accordance with studies of sunlight projection for buildings intended for human occupancy.

Front yards should be required in multiple-dwelling districts in order that living quarters may be removed from the noises of the street and from the dust and gasoline fumes caused by the larger volume of traffic usually incident to such districts. A front yard of 15 or 20 ft. would not seen unreasonable for a multiple-dwelling district for reasons previously mentioned, and incidentally would give opportunity for a lawn and planting at least suggestive of, and reasonably compatible with, a home. If the streets are narrow, front yards increase the opportunity for better air circulation and the degree of privacy within apartments fronting the street.

Front yards are desirable in single-dwelling districts because of lessened noises, dust, and fumes from the street, increased privacy, general amenity, and improved character, appearance, and stability of the neighborhood. A front yard of 20 or 25 ft. would seem to be a reasonable requirement and is the customary standard. Buildings on corner lots should recognize the front yards of side streets.

Side Yards.—Side yards should not be required in commercial or industrial districts. If side yards are provided and buildings or parts thereof are intended for human occupancy such yards should be sufficiently wide to admit of adequate light and air, based on studies of sunlight projection. The width of a side yard should be related to the height and length of the building and the minimum width should permit of easy access in case of fire. Narrow side yards are usually damp, dark, refuse collectors and, as such, are a menace to health.

Side yards should be provided on either side of multiple dwellings that have windows of individual apartments opening thereon. Access and egress in case of fire, privacy of apartments, and ample provision of light and air would justify widths of 12 ft. or more, depending on the design of the structure, as there is often little or no distinction between side or rear yards in multiple dwellings.

Side yards should be provided on either side of detached or semi-detached dwellings for access in case of fire, for privacy, and for general amenity,

character, appearance, and stability of the neighborhood. A minimum of 5 ft. should be required for sanitary reasons.

Rear Yards.—Rear yards should be required in commercial and industrial areas to permit within the interior of blocks an open space common to all structures. This would facilitate the provision of light and air to all structures intended for human occupancy. This is especially true where numerous buildings occupy a single block. Exceptional conditions often make impossible the realization of this ideal. Where alleys are provided and buildings generally seek light and air from the front and rear, this requirement is not impossible of realization. It is a more simple and satisfactory method than that of attempting to provide light and air for each structure on its own lot. Rear-yard dimensions should be based on sunlight projection and, consequently, should be proportionate to the building height.

Rear yards should be provided for all multiple dwellings, preferably of not less than 25 ft., to create adequate open spaces within block interiors for air circulation, privacy of apartments, and to facilitate access and egress in case of fire. Rear yards are needed for service entrances and garages. In intensively developed multiple-dwelling districts where people are crowded on the land it is often impossible for a city to provide adequate recreation spaces, and it would not be unreasonable to require a certain amount of open space in proportion to the families housed. This might be provided in the rear yard if not elsewhere on the lot.

Rear yards should be provided for single dwellings for reasons of privacy, light, and air, and for garage or accessory buildings. A rear yard of 25 ft. is a reasonable and customary standard in the average type of residential lot.

Courts.—Where provided to supply light and air to buildings intended for human occupancy, courts should be proportioned, in size, to the height of the building based on studies of sunlight projection.

Percentage of Lot Occupancy.—Opinions differ as to the desirability and effectiveness of regulations governing the percentage of lot area that may be covered by buildings. In New York, N. Y., this type of regulation has been useful in preserving the integrity and desirability of the single-family, detached dwelling districts. In cities where large lots are the custom, this type of regulation is more or less ineffective if not unreasonable. If used, this regulation should accompany yard and court regulations.

Population Density.—Families per Acre.—Control of population density, expressed in terms of families per acre, or square feet of lot area per family, is a most desirable form of area zoning. It prohibits overcrowding of land that is otherwise difficult to control. It encourages uniformity of land occupancy and, as such, should permit easy improvement of housing standards. It furnishes a logical point of departure for scientific city design, and a sound basis for utility design and for the construction of public improvements.

Objection has been expressed to population-density-regulation because of the varying sizes of "families", the unit of regulation. This is objection to method, not principle, and implies the need for a better form of expression or definition of terms. Public health, public safety, and public morality demand

certain control of density of population in accordance with the type of dwelling. Public convenience is substantially promoted thereby.

Whereas the limits of this paper do not permit of extended discussion of this subject, the regulations given in Table 1 will admit of good housing standards, although each should be accompanied by forms of open space regulations dependent on the particular design.

TABLE 1.

Type of building.	Square feet per family.	Number of families per acre.
Detached or semi-detached single-family dwellings	4 000	10
Single-family dwellings in rows or blocks	2 000	20
Single and double flats, 2 and 4-family (St. Louis, Mo., type)	2 000	20
Multiple dwellings	1 250 to 625	34 to 68

Conclusion.—The considerations given in this paper are offered more to suggest a basis for discussion of zoning for area than to recommend particular regulations. Each city offers a somewhat different problem in the social organization, habits, and customs of its people and form of dwelling construction. The form of zoning for area that will best promote good housing and living conditions should be studied for each city. Forms of housing that have proved their desirability should be preserved. New forms better adapted to modern conditions and to preserving good standards should be encouraged.

A wider understanding of the importance of minimum standards of good housing is needed. The honorable practice of land subdivision, improvement, development, and sale will thereby be promoted and dishonorable, wasteful, and inappropriate practices, discouraged and defeated.

THE PURPOSE OF ZONING ORDINANCES AND METHODS OF ADMINISTRATION

By Morris Knowles,* M. Am. Soc. C. E.

Synopsis

There is a growing appreciation and understanding by engineers of the importance of zoning regulations as a part of city planning. Much has been accomplished in zoning activities since January, 1923, and this paper attempts to call attention to some of the pitfalls and the dangers and the wisdom of a sane consideration of regulations before attempting to put them into effect.

The principal purpose, however, is to call attention to some of the administrative measures and to point out the lessons derived from some experiences, not only in the review and preparation of zoning regulations, but also in the actual operation and application of ordinances to given circumstances. Particular stress is laid on the necessity of starting right and this means: First, a comprehensive enabling act; second, careful study and preparation of data; third, thorough discussion with every one affected; fourth, the administration of such ordinances by capable and honest officials; and, fifth, the interpretation of special clauses in particular cases by a competent Board of Appeals, actuated by a sense of justice, fairness, and an humanitarian feeling toward all those who are affected.

Progress of Zoning

One of the sessions of the Annual Meeting of the Society in January, 1923, was devoted to the subject of City Planning, at which the speaker presented a paper entitled, "Zoning—Its Progress and Application".† At that time, attention was called to some of the necessary requirements in the production of a wisely drawn zoning ordinance.

Statistics.—At the beginning of 1923, zoning regulations were in effect in 129 municipalities and, in September, 1921, when the Advisory Committee on Zoning was organized in the U. S. Department of Commerce, only 48 cities and towns, with less than 11 000 000 inhabitants, had adopted zoning ordinances, and only 18 States had enabling acts. During the three years prior to July 1, 1924, zoning regulations had been adopted by 210 municipalities, the total number at present in the United States, being 258 cities and towns. Of these, 63 have a population of less than 5 000; 48 with more than 5 000 and less than 10 000; 40 with more than 10 000 and less than 25 000; 42 with more than 25 000 and less than 50 000; 47 with more than 50 000 and less than 250 000; and 18 have a population exceeding 250 000. The total number of people living in zoned communities is 24 000 000, or 44% of the urban population of the United States. All but seven States now have enabling acts in some form. It is of interest to

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† *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1349, *et seq.*

note the popularity of zoning regulations in the small towns, which frequently zone without competent advice and do the bizarre thing.

The statistics just given indicate the strong appeal which zoning has on the hearts of the people. There is a growing tendency, and an increasing belief on the part of all, that this expanding application of the police power has come to stay. There is a feeling on the part of the small home owner, that in no other way can ample protection be provided; on the part of the realtor, that wise and sane development can best take place; on the part of mortgage lending and insurance companies, that stability will be brought about; and on the part of city engineers, planners, and municipal executives, that only in this way, can the future development and needs of the city, with the necessary improvements and utilities, be provided without waste and with the prudent expenditure of public funds.

Official Education.—All this has been a matter of education and advancement of public sentiment which, in turn, is reflected in the decisions of the Courts. This naturally is to be expected in the interpretation of regulations on such a subject invoking the application of the police power. Much of this public education and sane development of zoning regulations has been brought about by the activities of Herbert Hoover, M. Am. Soc. C. E., Secretary of the Department of Commerce of the United States. On taking office, Mr. Hoover early realized that a proper labor of his Department would be the promulgation of information for the education of the people of urban communities regarding the wise exercise of their activities and prudent expenditures of money by thoughtful city planning, of which zoning regulations are a large part.

The publication and issuance of the Standard Enabling Act, in August, 1922, resulted in a large number of States either passing new Acts covering this subject, or making substantial amendments to several of the Acts already in existence. The recent publication of the pamphlet analyzing the zoning ordinances of the sixteen typical cities which had passed zoning ordinances prior to November, 1922, has resulted in bringing to the attention of many people, the variety of the regulations relating to use and height, which are the subjects thus far covered. It is to be expected that such information and the discussion therefrom will prevent many cities from attempting fanciful and visionary types of ordinances which are the results of personal whims or idiosyncrasies of local people. Such regulations are all too likely to be thrust aside by the highest Courts as an improper exercise of the police power.

Kind of Regulation Desired.—The type of restriction needed is that which, although not going as far as some would like, will still give to different groups of people the kind of control desired in the particular community. That which is sane and goes only so far will be a step of progress, but will not advance to an extent that is likely to be upset by the Courts. Zoning is a growing subject and as the people become familiar with the regulations, administrative officers become experienced in its operation, all become familiar with what can, and what cannot, be done; the Courts also will come to realize that every step is the result of careful consideration and a wholesome regard of the public need

and welfare. Then that stability which arises from certainty will be obtained, and the reversal and throwing into the scrap heap of certain types of attempts which have occurred too often in the past, will be obviated.

At one time the speaker made the assertion that:

"Zoning will stand or fall as a result of the care in its preparation and the degree of fairness used in its interpretation and appeal to all owners that it will produce beneficial results to the property of all as the years go by. Much, therefore, depends on its application and interpretation in a broad spirit without fear or favor during its early development."

An early Roman edict on stone tablets states that "The welfare of the people is the highest law". Both statements are as true now as when they were made.

Requirements for Good Administration

Although not strictly coming under the head of administration, it is quite as true in this field of endeavor as in every other, that a false start generally results in failure. As was said by Secretary Hoover in his foreword to the Standard Enabling Act, "The importance of this Standard State Zoning Enabling Act cannot well be over-emphasized."

Standard Enabling Act.—It is apparent, after careful consideration, that there is danger in not having a complete and comprehensive enabling act (and it is doubtful whether home rule city charters will ever be a sufficient substitute therefor). Such an enabling act should carefully state the grant of power in all detail; the method of dividing the municipality into districts; the procedure to be followed by the municipal legislature; the requirements as to careful, comprehensive, and detail study; the need of publication and hearings before adoption by municipal legislature; the provision for amendments and modifications, under careful restricted requirements as to notice and vote when certain percentages of the people protest; the provisions for a Board of Adjustment which shall have the power to vary the ordinance and grant exceptions in harmony with its general purpose and intent and in accordance with certain rules and under appropriate conditions and subject to the proper safeguards; also, the provision for remedies and penalties.

The importance of the grant of power is appreciated when it is realized that all the regulations as to use, height, area, and the details covering questions of occupancy and location; size of yards, courts, and other open spaces; percentage of lot occupancy and density of population, should be carefully stated in the Act. This is necessary in order to show that such limitations apply and are subservient for the purpose of promoting the health, safety, morals, or general welfare of the community. Section 3 of the Standard Enabling Act is quoted for the purpose of showing the necessity of exercising such care in drafting an ordinance:

"Section 3.—Purposes in View.—Such regulations shall be made in accordance with a comprehensive plan and designed to lessen congestion in the streets; to secure safety from fire, panic, and other dangers; to promote health and the general welfare; to provide adequate light and air; to prevent the over-

crowding of land; to avoid undue concentration of population; to facilitate the adequate provision of transportation, water, sewerage, schools, parks, and other public requirements. Such regulations shall be made with reasonable consideration, among other things, to the character of the district and its peculiar suitability for particular uses, and with a view to conserving the value of buildings and encouraging the most appropriate use of land throughout such municipality."

Study and Procedure.—Careful study is necessary in order that the division of the municipality into districts, of any desired size and shape, is best suited to carry out the purpose of the regulations. Such districts, however, should indicate, by their very expression, that they are uniform for each class or kind of buildings throughout the particular district. This does not mean that districts, particularly those required for use, may not be as small as desired, namely, as small as one block or the corner lot within the block, provided reason is exhibited in the specification of such regulation. This particular detail is important because of the all too frequent misunderstanding illustrated by the one item of use. Some have thought that the regulation with regard to residential districts of considerable size does not permit the introduction of the convenient neighborhood store. This, of course, is erroneous because, in some districts, certain areas, particularly corner lots, can be appropriately zoned for the neighborhood store with the proper height and area regulations.

Some municipalities have disregarded the important question of procedure and, therefore, it is necessary to state this quite carefully in the enabling act. Thus, anxious and hurried groups of people will not attempt to pass zoning regulations without careful study, without thorough-going discussion, and without having hearings, with sufficient notice in order that all can be advised. If there is any one thing that should be impressed on the advocates of zoning it is that much time must be given to public education and consideration.

One reason for the advisability and even necessity of public discussion is to avoid misunderstandings as to what zoning is for, how it originated, and how it really works. Much unfortunate, ill-advised, erroneous criticism has come from those who really wish to deceive because they do not believe in fair play. Lawrence Veiller, Secretary and Director of the National Housing Association, calls these "poison gas attacks". They relate to the assertion that zoning was "made in Germany", with the apparent feeling that such will damn it forever. The American people, however, are too intelligent to be moved by such assertions, even if true, because anything worth while from any country warrants thorough study and consideration to determine how far it may be applicable to American conditions.

The facts, however, are quite at variance. Zoning was practiced in France in 1810, at the time of Napoleon, more than half a century before it was applied in Germany. A recent Scandinavian student of the subject is said to have traced zoning to Italy and France as far back as the Fifteenth to Seventeenth Centuries. As early as 1667, the height of buildings was limited in Paris. Dr. Evans, of Chicago, Ill., states that the earliest known instances of zoning were the establishment of the ghettos in which the Jews were required

to live. These ghettos were districts outside the walls of the ancient cities. Zoning of to-day, however, is an American product suited to American conditions.

Amending Machinery.—The safety and stability of zoning regulations are dependent on the machinery by which amendments can be made. Zoning will always be a live subject, just as the growth of municipalities is a live and virile thing. There never will be stagnation; there should be no desire to prevent progress. Amendments to any such machinery of administration are likely to be desired and, in the progressive change and development of the municipality, it is evident that streets and districts will naturally change in character of use, calling for amendment. It should be possible, therefore, to have the provisions for making such amendments as reasonable as possible; but they should be safeguarded, also, so that, due to sudden whims or at the request of favored property holders, the desire to make amendments which may injure others, should not be too easily accomplished.

The well-established method of doing this is to provide that there shall be sufficient notice before an amendment can be considered by the legislative authorities of the municipality. Frequently, the body which first studies the zoning ordinance (be it a Planning Commission or a Special Zoning Commission) is called on first to consider such amendments and then report to the municipal legislature. In this way, there will be avoided that hasty and ill-considered action which fails to take into account the relation to other provisions of the ordinance and the effect on other properties in the district, or even on other districts. Then, again, if there are protests, it is necessary that a larger vote of the municipal authorities be required. Generally, a three-fourths vote of all members is required, in case there is protest of 20% of the owners of the area included within the enclosed change, or those immediately adjacent in the rear thereof, or directly opposite thereto, extending a certain number of feet from the width of the lots to be affected.

Board of Appeals.—Of all the rocks on which zoning has been shattered and become ineffective, the inability to have a Board of Adjustment (sometimes called Board of Appeals) is perhaps the one most frequently in evidence. A review of the decisions of the Appellate Courts, which have denied the application of zoning in particular cases shows that, although there has sometimes been lack of interpretation of the modern spirit of the application of the police power to this subject, in the majority of cases such adverse decisions have generally resulted from an inadequate enabling act. This has either not been sufficiently detailed and comprehensive, with sufficient safeguards, or has not provided this Board of Adjustment which, in its operation, permits a flexible and kindly modified regulation of so strict an instrument in a humane and appropriate way. No city is entirely free from special conditions, with peculiar topography and physical arrangements. Watercourses, hills, irregular shape of industrial districts, long time existing but incongruous business and residential districts, all call for special treatment with great consideration and care.

It is, therefore, proper and necessary that some body, other than the Courts, first be given the chance to review and to interpret, in necessary cases,

the application of the zoning ordinance where undue hardship and severity of the execution would ensue. Such Board of Adjustment should be authorized by the Enabling Act, in order that it may be legal for the municipal legislature to delegate such a grant of power. Otherwise, the municipal legislature is incompetent to extend such provision to any such Board. Such a Board may have as large a membership as seems necessary, generally from three to five members who should have rotating terms of office and may or may not be paid, as in each municipality seems wise. It should have certain stated fixed meetings and, although adopting rules for its own procedure, should be compelled to keep minutes of its proceedings, record votes, and have its records open to examination.

The Standard Enabling Act suggests that such Board shall have the following powers:

"1. To hear and decide appeals where it is alleged there is error in any order, requirement, decision, or determination made by an administrative official in the enforcement of this act or of any ordinance adopted pursuant thereto.

"2. To hear and decide special exceptions to the terms of the ordinance upon which such board is required to pass under such ordinance.

"3. To authorize upon appeal in specific cases such variance from the terms of the ordinance as will not be contrary to the public interest, where, owing to special conditions, a literal enforcement of the provisions of the ordinance will result in unnecessary hardship, and so that the spirit of the ordinance shall be observed and substantial justice done."

Such provisions do not mean that remedies are exhausted. There should also be the provision that an appeal will lie to any Court of competent jurisdiction from such Board of Adjustment. The operation of such a Board will materially relieve the duties of the Courts, and perhaps in 99 cases out of 100, will do away with the necessity of Court proceedings. If a case does reach the Court there is less likelihood of the provisions and principles of the zoning act and ordinance being stricken down, because it will be evident that at every step of the way the aggrieved person has had sufficient opportunity for a careful consideration of his particular case.

Remedies.—The enabling act should include the ordinary provisions for penalty for violation. These may not be a serious deterrent to a person desiring undue privileges. An additional power, therefore, should be granted to local authorities to institute appropriate action or legal proceedings to prevent the unlawful erection or construction, alteration, conversion, or use of any property in violation of the provisions of the zoning ordinance.

Administrative Agencies

It is customary to provide that the administration of the zoning ordinance shall fall on such municipal officer who has been in the habit of enforcing regulations regarding building construction, such as building codes, plumbing codes, fire ordinances, etc. In some cities, such officer may be called the Superintendent of Building Construction or the Building Inspector. In some communities, it may be the City Engineer or the Director of the Public Works,

and, in some smaller communities, it may be the officer in charge of Public Safety, or the head of the Police or Fire Bureau.

Enforcing Official.—In any case, it must be understood that such administrative officer has no power of discretion. He reads the ordinance and the maps which accompany it, which show, by suitable symbols and designations, the application of the text of the ordinance in certain districts of the city. His word is final, except for the ordinary provisions of Court appeal, as to the issuance of a permit. However, where provisions are made for the Board of Adjustment (and as before stated, this is an absolutely necessary provision), the decisions of this administrative officer are subject to review and change by the action of such Board. No longer does the decision of the executive officer become final and the Board always exists to modify or vary whatever he may have said.

Experience in Pittsburgh, Pa., shows that, almost without exception, a denial of the application of the Superintendent of Building Inspection results in an appeal. Whether this will always continue, with a growing understanding and appreciation of the ordinance, is yet to be determined; but at least in the early experience of the ordinance, this seems to be the natural and expected thing.

In the early period of the administration it is to be anticipated that such executive officer and his force will not have the knowledge of the details of the ordinance, its intent, and its application, as thoroughly as they will after it has been in use for some time. It is to be expected, therefore, that some mistakes will be made, that the owner will not take "No" for an answer. Until he learns the spirit of the Board of Appeals and its method of operation, he will always try to have such an order changed or modified. Real estate operators, contractors, and owners will all learn in time just what can be accomplished under the law and in certain districts. It is quite possible, therefore, that in the future there may be less work for the Board of Appeals.

Effect on Building.—It will be of interest, in this connection, to review the results of the operation of the zoning ordinance in Pittsburgh, as exemplified by certain statistics, which include the activities of the Board of Appeals. The zoning ordinance of Pittsburgh, which city has a population of about 626 000, became effective on August 9, 1923, and the Board of Appeals was appointed and began its activities on September 5, 1923. Up to August 1, 1924, the Board of Appeals had received 964 applications, of which 889 have been acted on, in one way or another. The City of New York, with a population of 6 000 000, had 624 cases in the calendar year 1923, of which 167 remained undetermined on December 31, 1923, leaving 457 as acted upon.

In Table 2, there is presented the total permits granted for the twelve months prior to August 1, 1923, and for the period ending July 31, 1924. This also shows permits for new dwellings. Table 2 also shows the number of cases presented to the Board of Appeals for each of the twelve months ending July 31, 1924. Of these appeals occurring within the twelve months, 635 affected use, 11 affected height, 310 affected area, and 8 affected a combination of two or more of these restrictions.

TABLE 2.—Comparison of Building Permits in Pittsburgh, Pa., Before and After Passage of Zoning Ordinance.

Months.	Building Permits.				Appeals.	
	August, 1922-July, 1923.		August, 1923-July, 1924.		August, 1923, to July, 1924.	Percentage of appeals.
	Total.	Dwellings.	Total.	Dwellings.		
August	590	63	557	132		
September	542	135	491	92	49	10.0
October	561	122	656	167	67	10.2
November	437	110	545	174	65	11.9
December	313	114	500	158	72	14.4
January	292	89	355	120	70	19.8
February	336	129	424	144	70	16.5
March	688	271	651	216	71	10.9
April	836	241	911	240	140	15.4
May	811	193	945	216	124	13.1
June	713	157	909	179	104	11.4
July	755	142	788	166	132	16.8
Total	6 874	1 766	7 732	2 004	964	12.4
Percentage increase	.....	.....	+ 12.5	+ 13.5	.....	.....
*Permits in 65 cities	353 250	186 212	402 122	208 891	.....	.....
Percentage increase	.....	.....	+ 13.8	+ 11.8	.....	.....

* Of 68 cities having population of 100 000 or more, July, 1923, to June, 1924, inclusive.

Table 3 shows the disposition of all appeals for the year ending July 31, 1924. It is apparent that by far the greater number of appeals are granted either completely or with conditions.

Because of the peculiar topographical conditions in Pittsburgh, the limited area devoted to business within the triangle bounded by two rivers and Grant's Hill and the congested river-bank area devoted to industry, it is apparent that there will always arise a number of cases which must be reviewed by such a Board of Appeals.

Character of Amendments.—The question of changes in amendments has already been thoroughly discussed, but a brief review of the indications of that which may happen generally, even with the most carefully drawn ordinance, may not be amiss. It must be taken for granted that, with the greatest discussion and publicity that can be obtained, zoning ordinances will be passed that are not understood by a large number of people. As soon as the application is felt and the pressure applied, many will object and feel aggrieved; thus there early arises a desire for several types of amendments.

In almost all cases these early requests arise through misunderstanding and inability to appreciate what can be accomplished; sometimes from a desire to accomplish anything that personal whims dictate; sometimes this comes from a lack of public spirit and unwillingness to estimate properly the ill effect on adjoining owners and sometimes from purely mercenary motives and the desire to get all the money possible from a given investment. This results

in requests to the friendly councilman to introduce an amendment to correct the supposed error.

TABLE 3.—Disposition of Appeal Cases in Pittsburgh, Pa., from August 1, 1923, to July 31, 1924.

Months.	Total appealed.	Granted.	Granted with condition.	Denied.	Held over.	Revoked.	Withdrawn.
August	....	....	....	....	....	....	....
September	49	7	7	9	26	....	....
October	67	10	30	21	31	1	....
November	65	10	38	20	27	....	1
December	72	13	24	21	40	....	1
January	70	20	26	23	41	....	....
February	70	19	35	15	41	....	1
March	71	20	32	23	37	....	....
April	140	36	57	26	57	....	1
May	124	34	62	26	59	....	....
June	104	24	46	31	62	....	....
July	132	22	60	37	75	....	....
Total	964	215	417	252	75	1	4
Percentage of total		22.3	43.2	26.2	7.8	0.1	0.4

During the early history of the zoning ordinance it would be well if as few amendments as possible are presented, or if they are presented, that they receive such careful and long-time consideration as will prevent stampeding. If too many amendments are made at once, the impression is likely to be created that zoning regulations are ephemeral, that they are due to immature consideration, and that they ought to be wiped out wherever they run counter to private desire. The usual machinery for amendment is such, of course, as to prevent hasty consideration, but even with such precautions, some amendments will be requested which should never have been presented. The longest time history of a zoning ordinance on record is that of New York, which was passed in 1916, and the story of adopted amendments, which is presented in Table 4, is interesting.

Table 4 shows that, for the first four years during which the zoning ordinance was in effect, the amendments passed were largely those of a nature to lessen restriction. Beginning in 1920, the restrictive amendments have been increasing both in number and in percentage, until at present (1924), they outnumber the relaxing ones three to one. Perhaps this is the natural development to be expected. If an ordinance has been carefully drawn and wisely administered, the people will appreciate its beneficial effects. Although there may be some tendency in the beginning to object, the general pressure of public opinion which is effective and which appreciates stability and desires protection, will see to it that only those amendments will be passed which are helpful and in sympathy with the spirit of the ordinance. It is too soon, from a statistical point of view, to determine the result in Pittsburgh, but it is believed that the effect will be similar. Only twelve amendments to the Pittsburgh zone map have thus far been presented.

TABLE 4.—Changes Granted by Years in Greater New York Zoning Plan.*

Year.	Number of changes.	Relaxing Changes.		Strengthening Changes.		Ratio: Strengthening to Relaxing Changes.
		Number.	Percentage.	Number.	Percentage.	Percentage.
1916	4	4	100	0	0	0
1917	43	36	84	7	16	19
1918	26	20	77	6	23	30
1919	20	13	65	7	35	54
1920	27	12	44	15	56	125
1921	38	15	39	23	61	153
1922	61	14	23	47	77	336
1923	81	19	23	62	77	326
Total..	300	133	44	167	56	126

* Some amendments include more than one change.

Results from Application

It was stated during the consideration of the Pittsburgh zoning ordinance that the passage of such regulations would have an adverse effect on building operation. A study of Table 2 indicates the contrary. The total building permits during the last half of 1923 and the first of 1924—following the adoption of the zoning ordinance—exceeded the total during the preceding year by 12.5%; and those for dwellings by 13.5 per cent. For comparison with general conditions throughout the country, the permits for 65 of 68 cities having a population of 100 000 or more—as recorded by the Department of Labor—have been studied. The increase during the period from the middle of 1923 to the middle of 1924, over the preceding year, was 13.8% for the total and 11.8% for residences. It will be noted that, in general construction, Pittsburgh differed little from the general average throughout the country, but showed a rather marked increase in the construction of new dwellings. This would indicate confidence in the protection of home neighborhoods by the zoning regulations. In fact, there is a growing belief in the city that such ordinance has come to stay; that it is helpful rather than damaging to wise building activity; and that property values have been stabilized. The ordinance has probably won friends rather than made enemies through its administration.

Personnel of Appeal Board.—The policy and methods of the Appeal Board may have contributed somewhat to this situation. The Board is composed of three members appointed by the Mayor. One, the manager of one of the four large estate holdings in the city, which owns large unimproved as well as improved land, represents the real estate interests and brings to the Board a sane and conservative view with regard to the uses of property. Another member is the Secretary of the Retail Merchants Association; he is also a member of the Transit Commission, Secretary of the Flood Commission, and has had the experience of broad public connections. The third, a member of the City Planning Commission, and also a real estate operator, acquainted with conditions which obtain with the small home owner and renter,

is familiar with the drafting of the ordinances, the consideration given to the various parts, and the public discussion given to each. Thus, the Board, wisely constituted and actuated by earnest purposes, considers every question on its merits and with a friendly spirit of enforcing the ordinance without undue harshness.

This Appeal Board started with the idea of visiting every property on which an appeal was taken, and from the beginning has held weekly sessions. The number of cases has varied from fifteen to thirty-five each week. Monday afternoon is devoted to hearings, at which the testimony of the applicant and protestants is taken. Wednesday is devoted to consideration and dictation of the decisions and re-hearings, if any. Friday afternoon is devoted to a consideration and signing of the decisions as written and the listing of cases to be heard on the next Monday. Visitations are made after such listing and before the case is heard on the following Monday.

Court Appeals.—Only four cases have been appealed to the Courts in Pittsburgh, and decisions on two have been reached. The first case is one of an appeal by certain property owners against the granting, after review by the Board of Appeals, of a permit for the construction and operation of an incinerating plant in a light industrial district. It should be stated that the ordinance provides that such authority to issue a permit is granted to the Board of Appeals, but not to the Superintendent of the Bureau of Building Inspection, under the following listed powers of the Board of Appeals: "For the use of land and the erection and alteration of buildings in a Heavy or Light Industrial District for the incineration of refuse or rubbish in properly constructed furnaces." After careful study on appeal from the denial by the Superintendent of Building Inspection, the Board of Appeals caused a permit to be issued for the erection of such a building with the following conditions:

"1.—That the owner provide on its own land sufficient storage facilities for twenty (20) truck loads of material, so that there will not be any occasion for the stopping of trucks in Juniper or other streets in the neighborhood of the premises while awaiting turn to be unloaded.

"2.—That nothing in this permit shall carry the right to use for any purposes the land within the lines of Juniper and Neville Streets as located at the front and rear of the property.

"3.—That the unloading, receiving, and charging facilities be under cover and be such as to prevent nuisance, uncleanliness or dust, or the accumulation of material, débris, or ashes; and that suitable ventilation be provided, so that dust which comes from unloading can be taken through into the settling chambers, or stack, by means of suction fans or otherwise.

"4.—That the furnaces be sufficient to consume the maximum amounts of material rapidly and at temperatures sufficiently high to prevent odors and with sufficient size of dusts, settling chambers, and stack to have a slow enough velocity and with stack high enough to produce the necessary draft and not carry odors, gases, or dust to the top thereof and discharge over the neighborhood."

On the grant of such permit on appeal the aggrieved property owners in the vicinity took the case to the Common Pleas Court of Allegheny County on the plea that:

"(1) The decision of the Board is illegal on the grounds that the Acts of Assembly and the Ordinance of the City of Pittsburgh are unconstitutional and void;

"(2) The decision of the Board of Appeals violates the provision of the Act creating the Board of Appeals;

"(3) The decision is in violation of the sections of the ordinance which prohibits incinerating plants in light manufacturing districts;

"(4) The Board of Appeals had no jurisdiction to grant permit until plans and specifications of the proposed incinerating plant had been filed, showing the exact type, style, location, kind of material, and furnace to be used;

"(5) The Board heard about three or four witnesses of the objectors, that twenty were present and ready to testify. This decision is arbitrary and against the weight of the evidence."

The Court decided on February 12, 1924, (1) that grounds of illegality specified by the objector should not be sustained; and (2) that the Board acted in compliance with the ordinance legally adopted in pursuance of legislative authority and exercised legal power given it by the ordinance. This case has been appealed to the Supreme Court of Pennsylvania.

The second case, which is somewhat more complicated, arises out of the illegal act of the petitioner in enclosing a front porch on his house. The definitions of the ordinance provide that a porch is a roofed open structure, projecting from the front, side, or rear wall of the building. A front yard is one extending across the full width of the lot, from the front line of the building to the front line of the lot. Thus, an enclosed porch becomes part of the building, and the depth of the front yard must then be measured from the face of the projecting front porch which has become the front wall of the building.

The owner, without securing a permit, proceeded to enclose his porch. Various complications and misunderstandings arose because of this illegal act. Proceedings were entered to fine him for beginning construction without securing a permit, but later these were held over, pending disposition of the greater question arising out of having done a specified act prohibited by the zoning ordinance. After the structure had been completed, and in order to validate his action, the owner applied for a permit. It was refused, and the matter proceeded to the Board of Appeals, as provided for in the ordinance. The Board sustained the Superintendent of the Bureau of Building Inspection and refused to order the issuance of the permit. This case was taken to the Common Pleas Court of Allegheny County and decided June 9, 1924.

The particular section of the ordinance which was violated was the one requiring that:

"When the front wall of eighty per cent. of all of the buildings on one side of the street, between two intersecting streets, has been kept back from the street line, no building hereafter erected or altered shall be placed nearer to the street line than the distance established by the majority of the eighty per cent. at the time of the passage of this ordinance."

Although some of the houses within the block had a lesser depth of front yard than that which obtained when this porch had been enclosed, it was

apparent that, in this particular case, the depth of front yard would not conform to the requirements of this 80% clause. The appellant in this case alleged, as usual, violation of the Constitution of the State and illegality of the ordinance, and particularly that no power existed to prevent the enclosing of the open porch already constructed.

The Court decided that the enclosed porch, as constructed, had been turned into a part of the house, and its construction was prohibited by the ordinance. The Court also decided that the constitutionality of the Enabling Act had been approved by the same Court in the case previously referred to and, further, that:

"The requirements of conformity of depth of front yard to that already put in use by eighty per cent. of the owners was not unreasonable and in fact it would be difficult to imagine what regulation could be made on that subject which would be more favorable to individual owners and at the same time amount to anything as a regulation."

This case has been appealed to the Superior Court of the State of Pennsylvania and will probably be considered after the Supreme Court has decided the former case.

Experience with Zoning

Additional Problems.—Experience has shown that there are many details of zoning on which additional enlightenment is needed and which should be studied. Perhaps the most important is that of the regulation of the height of buildings. A reasonable recognition that the skyscraper is not alone responsible for all forms of street congestion will go far toward obtaining a wise interpretation of conditions as they are found.

Skyscrapers.—In American cities, the location of theaters is not controlled as in London. Consequently, as exhibited in New York, most of the theaters empty their population at one time and within such a small area that the capacity of the few stations on the subway in the vicinity is taxed to the utmost for a half-hour period following the close of the theaters. A further recognition that large department stores, many of them within short distances of each other, materially serve to congest the sidewalk population and even the roadways, to a large extent, and greatly add to the fire hazard, will do much to aid in understanding that the problem is not simple.

In this country the skyscraper has a great appeal, and our large buildings have no equals anywhere else in the world. Whether it is due to a question of personal pride, or a desire to get ahead of the other fellow, or to make a monument for one's name, it must be evident that both private and public money is being wasted. As a prominent real estate authority in Chicago has stated:

"A tall building is responsible for raising ground value above the possibility of its earning power. The skyscraper in Chicago is an economic and financial blunder."

With the accomplishments of New York and the absurdly high limits of Chicago and Pittsburgh of 264 and 265 ft., respectively, the average height

of buildings in Manhattan is only 4.8 stories, and below Canal Street, only 7.8 stories. In Chicago, within the Loop, the average height is about 7 stories. In the business triangle of Pittsburgh, between the two rivers, the average height is 4 stories. It appears, therefore, that even with such liberal possibilities, not every one wishes to be foolish, but many are driven beyond a reasonable performance by the avarice of their neighbors. Public sentiment grows with popular education. As Mr. Veiller has expressed it:

"The private financial interests * * * would do well to give careful heed to whether such comparatively mild limitations on height as are proposed in most of our zoning ordinances, will be enacted, or shall we have in a few years in the United States what they have had in England for many centuries, 'A Law of Ancient Lights', or its equivalent, by which a man who erects that which diminishes in any way the light of his neighbor, will have to pay substantial damages for the impairment that he has wrought to neighboring property. That is the law in England to-day and has been since time immemorial, and because of that law the great City of London is free from the skyscraper."

Single-Family Districts.—One of the mooted questions in zoning to-day is that of prescribing single-family districts in certain sections of the city. It is certain that unless the Enabling Act is quite definite in its statement, it is extremely dangerous to legislate for one-family districts. Even then, it may be better to accomplish this purpose indirectly by such provisions as regulating the number of families per acre or the size of lot per family.

However, there is a growing desire on the part of many that one of the principal functions of zoning is to preserve the homelike character of single-family residential districts in many cities and this does not emanate altogether from the well-to-do. Even the small wage earner likes his little cottage with plenty of land, when he can afford it, and does not enjoy the encroachment of the apartment building or the row house. There are many reasons for this, and much can be said in favor of this extension of the police power to promote the safety, health, morals, and general welfare. There will be fewer delivery wagons, fewer store men and attendants; less noise, dust, and danger to children on the streets; less opportunity for the incipient cause and spread of fire, and many similar reasons which have been recognized in some jurisdiction by Courts of the last resort.

All is not clear, however, and much progress must be made, facts ascertained, and the public and Courts educated, before it can be certain that in all cases there will be a uniform upholding of such restrictions as one-family districts, particularly as compared to the double house and the two-family house.

Front-Yard Depths.—Another subject which has led to much controversy is that of the requirement of the depth of front yard, particularly as relating to the encroachment of porches, covered and uncovered, enclosed and unenclosed. Reference has been made to the White case in Pittsburgh, which is still to be heard in the highest Courts of Pennsylvania. Careful distinction must be made between the requirements of the depth of front yard and the reasons therefor, and the customary and frequently observed regulation of building set-back lines which, in some instances, have been established for an

entirely different purpose than those arising under the zoning law. The speaker refers to the desire to have buildings kept back on opposite sides of streets, so that ultimately a street may be widened without the payment of excessive damages for structures and by the payment only for the land taken. This method of widening streets and serving notice some years ahead of the actual operation has been a common practice in some States.

However, such action must proceed under the authority of eminent domain and when the land is actually taken, compensation must be paid. The preliminary location of the street line is only a warning to the owner not to erect buildings. The regulations of zoning, however, and the provision for a sufficient depth of front yard, or may be the width of side yard at corners, must be sustained under the police power, on the basis of promoting health, safety, and general welfare.

Stability Desired.—Presumably, there will always be some discussion as to that which is fair and reasonable in zoning. In time, stability will be acquired. This may be a slow process and require much education. Something will depend on the temperament of the people in a given locality, and how much they have been hurt in the past by the adverse action of others. One thing is certain, however, popularity is a dangerous thing, and as to zoning there is no exception. It behooves those who are sincerely in earnest, therefore, to go slowly and see that too rapid strides are not made too far in advance of the general public necessity and desires. There has been considerable campaigning among certain selfish interests inimical to the understanding or an intelligent discussion of the subject of zoning, but this is now abating, as a result of quiet but persistent and intelligent discussion of the principles and the need. It is interesting to quote from remarks of the General Solicitor of the Title Guarantee and Trust Company of New York, the following:*

"The highest form of ownership of anything, personal property or land, would be unlimited in duration and unfettered by any limitations upon use; but there is no such thing in any civilized community. All property is liable, in every civilized community, to those limitations upon its use or ownership which the necessities of civilized life impose, as expressed in the law. * * *

"Under police power, the community may restrict the use of property to avoid hurting the life, health, or morals of others. The community may establish and enforce ordinances to provide a certain standard of light and ventilation, to prescribe how buildings may be erected and equipped, and the percentage of ground to be covered. And if these requirements be not observed, the State will circumscribe the owner's dominion over his land. * * * Those who are fighting City Planning, of which Zoning is the cornerstone, seem to labor under the delusion that the owner of land has the absolute title. The above shows very clearly that our rights are limited. And for the sake of the best interests of society they perforce must be limited. * * * So, as a matter of fact, the principle is already established that an inappropriate use of land may be prohibited—therefore, why fight Zoning, which is the most orderly way of obtaining the desired results."

J. C. Nichols, Realtor, of Kansas City, Mo., than whom there has been no more advanced civicist and public-spirited real estate man in the country, has put it directly up to his professional brethren when he asks:

* From *Magazine of Building Management*, February, 1924.

"Is your city poorly zoned as to use, height, and bulk of buildings?

"Do you, in your city, permit property values to be stolen by unnecessary encroachment of property of injurious uses?

"Have you carefully studied the proper building heights for your particular city? One tall building in New York to-day casts a shadow at 4 P. M. across 7½ acres of land, taking away the essential sunlight from all the occupants of the buildings within this area.

"There is both a human and commercial limit to the carrying capacity of streets serving the pedestrian and vehicular traffic incidental to such tall buildings. * * *

"Are real estate values stable in your city, or do they shift almost with each decade? Have you blighted residential areas, or abandoned business sections? Is your city building a constant process of building up and tearing down—an extravagant American custom and a gigantic economic loss?"

Walter Stabler, Comptroller of the Metropolitan Life Insurance Company, of New York, says that an estimate of $1 000 000 000 per year is conservative for the losses resulting in American cities as a result of shifting residential and business sections and the economic hazards and property injury resulting from improper encroachment of enterprises of undesirable and conflicting character.

Summary

Much progress has been made in the field of zoning since the City Planning Division of this Society was organized. Popular sentiment, which has been developed by well-intentioned and well-informed publicity, resulting in more thorough education, has had much to do with this.

However, failures are still being recorded. In general, these are due to an incomplete or inadequate zoning enabling act; to the lack of comprehensive and thorough local study; too much haste and lack of publicity and education; partial, piecemeal, temporary, whimsical, or unreasonable regulations; too difficult or too easy provisions for securing amendments; and to the fact that there is no Board of Adjustment, to vary the ordinance in special cases of difficult interpretation.

The whole atmosphere of zoning regulations, their inauguration, preparation, administration, and interpretation should be one of reasonableness and sanity, with no attempt to accomplish a result too far in advance of the general public sentiment. Interpretation of such a drastic instrument should be in a kindly manner, with a firm yet friendly spirit.

Many unsolved questions still remain and many such as the proper restriction on height of buildings and the effect of such on congestion and safety, and the extent to which one should go in prescribing single-family houses and strenuous area restrictions, are matters about which much more information may be expected. In all stages and in all progress, the fact must not be overlooked that zoning is a part of city planning and not a thing separate therefrom. Bacon says:

"He that builds a fair house upon an ill seat, committeth himself to prison. Neither is it an ill seat where there is ill air only, but where there are ill ways, and ill neighbors."

HEIGHT LIMITATIONS IN ZONING

By Jacob L. Crane, Jr.,* M. Am. Soc. C. E.

The earliest limitations on building height, as far as the writer has been able to determine, were established in Rome under the Emperors Augustus and Trajan. The congestion of population in Rome, as in modern cities, made the high tenement house profitable. Acting under the pressure of complaint against the overcrowding, with its fire hazard, Augustus limited the height of all new buildings to 70 ft., and Trajan established a 60-ft. limitation.† The need for regulation and the precedent for it are, therefore, not less than 1 500 years old.

In modern times the precedent for height limitation was established in France and Germany, and from there spread to the Swiss, Scandinavian, English, and American cities. In 1882, height regulations were in effect in Paris. In 1891, shortly after he had been employed as Mayor of Frankfort, the famous Franz Adickes introduced a zoning system which incorporated height regulations expressed in terms of the street width.

The London Building Act, which regulates the height of buildings, went into effect in 1894. An unsuccessful attempt was made during 1923 to break down this code and to permit "American building development". The limitations in European cities range from 43 ft. in Zurich to 80 and 82 ft. in London and Vienna, respectively.

In America, the first height limitation was established by the City of Chicago, Ill. In 1892, an ordinance was passed setting 150 ft. as the maximum height, but this ordinance was vetoed by the Mayor. During the following year an ordinance went into effect limiting the height of buildings to 130 ft. After being raised to 260 ft., reduced again to 200 ft., and again raised to 260 ft., the Chicago height regulation has finally been incorporated in a comprehensive zoning ordinance (1923) which sets the limit at 264 ft. on the building line, plus additional height with set-back.

Several other American cities established height limits previous to the era of zoning, as follows: Baltimore, Md., 175 ft.; Boston, Mass., 125 ft.; Erie, Pa., 200 ft.; Los Angeles, Calif., 150 ft.; Milwaukee, Wis., 225 ft.; New Orleans, La., 160 ft.; Portland, Ore., 160 ft.; Washington, D. C., 160 ft.; and Minneapolis, Minn., 185 ft.

In all these larger cities (no less than one hundred in Europe and America), the conditions making height limitations necessary have been much the same. Laws of this kind, limiting the use of private property, have never been enacted easily, and height regulations could not have been passed except with the support of purposes of great public importance. Communities have found it necessary to curb the greed and the disregard for the public interest on the part of property owners. The theory underlying height limitations as important measures for public welfare may be outlined as follows:

* Municipal Development Engr., Chicago, Ill.

† *The Town Planning Review*, London, England.

(*a*).—Such regulations control the fire hazard, which increases with each additional story, other conditions being the same.

(*b*).—They tend to limit the congestion on the streets and in public conveyances, the congestion which follows the crowding of people and business in buildings served by the streets and public conveyances.

(*c*).—They help to preserve the light and air necessary to healthful life in crowded centers.

(*d*).—They spread business and, consequently, values over a larger area than would be occupied by higher buildings.

(*e*).—They have a favorable effect upon architecture, tending to produce a uniform cornice height along streets.

The advantages of high buildings which have been urged as arguments against height limitations, are:

(*f*).—They bring great numbers of people within a small area and thereby make it possible to go quickly and conveniently from home to workshop, or from office to office, to the end of promoting great efficiency in a city's business.

(*g*).—They make it possible to earn a return on enormous land values, which produce taxes, as do the great buildings themselves.

(*h*).—They are in themselves clean, healthful buildings, particularly for office purposes.

(*i*).—They offer opportunity for monumental treatment as an expression of American life, which can only be shown by the steel frame skyscraper; and these monuments cannot be simply ornamental towers, they must be big enough to "pay."

Based on the theory just outlined, municipal legislative bodies in America have found it to be sound public policy to establish "reasonable" height limitations under the police power which is vested in the States and delegated by them to the cities and villages. Furthermore, the Courts have sustained such applications of the public power in several conclusive cases. The earliest test case was in Boston, Welch *vs.* Wasey, in which the height limitations were upheld by the highest Court in Massachusetts and, later, by the Supreme Court of the United States. A recent and perhaps the most conclusive case is that of the Wisconsin Telephone Company *vs.* the State of Wisconsin, in which the Supreme Court of Wisconsin upheld the height regulations of the Milwaukee zoning ordinance. There seems to be no room for doubt that height regulations reasonably applied are legally sound. The problem before the bodies enacting such regulations is to secure the maximum effectiveness within the limitation of reasonableness in any given circumstances.

In this country, height regulations are now usually established as parts of comprehensive zoning plans which regulate the use of private land, the area which may be occupied, and the height to which buildings may be constructed in districts of different classes. The heights permitted by regulations range from the maximum for office buildings in the central business districts, lesser heights for apartment houses, local business and industrial districts, to the least for districts in which only single-family houses, churches, schools, and accessory uses are permitted.

To attain ideal conditions in the building of a city, the allowable height would differ for different kinds of buildings. Furthermore, ideal regulations for height would vary with the different objectives for which they are intended. These ideal height limitations may be compared with the limitations of American zoning ordinances, in order to determine the relative effectiveness of such ordinances and the possibility for improving them within the range of reasonableness.

Fire Hazard.—Taking the objectives of zoning in the order in which they have just been named, the ideal limitation of the height of buildings to control the fire hazard may be first set down. No frame building should be more than 2 or 2½ stories high. To meet this requirement, the building code, which must dovetail with the zoning ordinance for the sake of the effectiveness of both, should establish the character of fire-resistant construction required where buildings more than 2½ stories in height are allowed. The problem of height limitation, therefore, as regards fire hazard, is that of the so-called fire-proof building. How high can fire-proof buildings be constructed with safety to the occupants? The following is quoted from John Plant, Chief Engineer, Bureau of Fire Prevention and Public Safety, City of Chicago:

"The range of operation of modern fire departments is six or seven stories from the ground. Above this height we are dependent upon inside protection. Many years' experience in the inspection of inside fire-fighting equipment brings out the fact that this interior protection is rarely kept up to fire department ideas as to what this equipment should be.

"The matter of outside protection on window openings, whether this protection be in the shape of metal frame and wire glass windows, rolling steel shutters, or the ordinary iron shutters, or an outside water curtain, has also proven faulty. This is so because metal frame and wire glass windows will resist a temperature of only about 1 500° Fahr. Temperatures developed within a burning building usually grow to 2 200°, at which heat, brass fuses. Brass fused in the Burlington Building fire, and wire glass windows ran like molasses. The question of upkeep of rolling steel shutters, iron shutters, or outside sprinklers, introduces quite a problem because of the fact that these devices are of metal and are in the open atmosphere exposed to the rain and elements. Also, in this particular locality, there is so much soft coal burned, which contains a large amount of sulphur that gives off various corrosive gases which readily attack all metal exposed; thus, in a short time, your exposed metal devices are soon beyond repair.

"It is true that many buildings are equipped with outside iron fire escapes. This, in itself, is a confession that there is danger in the building. A fire escape is useful to an acrobat or a trapeze performer. The average citizen would be helpless descending from the upper floors on our stairway fire escapes. And in the winter time when these stairways are covered with snow and ice, they are positively dangerous. Fire escapes permit greater floor areas without taking out space for interior stairways, and thus are great favorites for increasing the revenue from a given building. In this same line, might also be mentioned the progressive building manager or owner who decides that the interior stairways are being wasted and who cuts off or takes out entirely an interior stairway in order that floor areas may be increased. The problem then of getting that stairway placed back is like asking the gentleman for his right eye.

"Fire escapes in times of fire with window openings directly underneath, as they of necessity must be, are soon put out of service by smoke and fire

coming up. It is not unusual for our firemen working on a fire escape to have the fire break out underneath, and large volumes of smoke and fire prevent their coming down the fire escape, making them ascend the fire escape to the roof, where they must find other means of getting down.

"I believe I am safe in saying that nearly all the occupants of an office building do not know where the stairways or fire escapes are. This is so because every day they depend upon the elevators to take them to and from their offices. Should a fire or disaster occur the first thing that goes out of service is the elevator. Now, where are the stairs and elevators? Are they fit to be used or are they, too, out of service because of fire, smoke, or gases, or whatever may be present?

"I do not hesitate to say that had the fire in the Consumers Building, which occurred at 4:10 A. M., on the morning of July 1, 1917, occurred during the hours from 9 A. M. to 5 P. M., Chicago would have seen a holocaust far more horrible than either the Iroquois Theater disaster or the Eastland boat disaster. No one, absolutely no one, and I say this without fear of contradiction, would have left the Consumers Building above the fourth floor, while the fire raged an hour and ten minutes beyond our control. And why beyond our control? Simply because the temperature reached 2 200°. Water is used to extinguish fire. Water boils at 212° Fahr. Chemists tell us that along about 1 200° or 1 400° water is broken into its constituent elements, hydrogen and oxygen, which substances only feed the fire. What is our practice? All we do is try to cool the fire and prevent its spread into adjoining structures. In this particular fire, it crossed a 16-ft. alley and inflicted damages totaling about $150 000 in the Baltimore Building.

"At the time of fire in a large office building when the body of fire on any floor assumes such proportions that it is leaping out of all the windows, at least five hose streams are necessary on each floor. Take, for example, where the upper eight or nine floors are involved, as was the case in the Burlington Building. This means 40 or 45 hose streams, meaning 4 or 5 fire engine companies in the building. This is nearly half of our total strength in fire engine companies. It takes quite a time to assemble this amount of fire-fighting equipment. In the meantime, the fire is growing, it is not waiting.

"We have had many serious fires in office buildings in Chicago, where the fire originated in the building itself and spread from floor to floor, out through the windows on one floor and 'mushroom', as we say, into the windows on the floors above.

"Fires in fire-proof office buildings are usually very hot, and the blast of heat and unburned gases is always projected horizontally outward from the building. This is so because the floors are usually of concrete or tile and do not permit the hot air and gases to go up through the floor.

"Standpipe equipment in a building is taxed to its utmost to hold a small fire on one floor, because only one or two lines can be taken from the standpipe as only one or two lines are supplying the standpipe from the Siamese connection on the street level."

W. D. Mathews, Chief Engineer, Chicago Board of Fire Underwriters, states:

"In reply to your inquiry regarding the height of buildings that should be permitted in Chicago, I must say that I cannot give you a definite answer without first knowing or assuming certain important conditions, *viz.*, construction, protection, occupancy, exposures.

"If you were to base the height on the ability of the public Fire Department to cope with the fire from the ground, then you should make the limit in Chicago at the present time five stories, or, say, 65 ft.; but if the Fire Department were fully equipped with water towers and apparatus essential for

delivering large streams at high pressure at maximum height, you might build as high as ten stories, or, say, 125 ft."

Sidney J. Williams, Chief Engineer, National Safety Council, writes:

"Aside from the health hazards which have already been discussed, high buildings involve two outstanding dangers to life.

"(*a*) Possibility of a serious loss of life in an office building fire. The following facts are well known to every one familiar with fire protection:

"1.—Fire-resistant construction will not prevent the contents of a building being completely destroyed by fire originating either outside (as in the Burlington fire) or inside (as in the old Equitable Building fire). There are numerous examples of both kinds.

"2.—This is true even where the combustible contents are no worse than in ordinary office occupancy, as in the two examples cited. Where an office building is eventually used for mercantile purposes, as is often the case, the hazard is much greater.

"3.—Fire-proof enclosure of stair and elevator shaft is very desirable, and often makes these exits usable during a fire, but cannot be depended upon for absolute safety in a high building, because a single door left open, or a small crack, may permit such shafts to become filled with smoke (or fire) rendering them impassable. Also, elevator machinery may be put out of commission by a fire, and it is physically impossible for the average person to walk down twenty or thirty flights of stairs without stopping to rest.

"4.—Outside 'fire escapes' are useful for the Fire Department, but practically valueless as a means of escape on high buildings. The City Fire Department cannot operate effectively from the outside above the eighth floor or thereabouts.

"5.—Inside stand-pipes are often improperly installed, and more often improperly maintained. Many, and probably a majority, of existing installations are of little or no use in an emergency.

"6.—We have not yet had an office building disaster, with the loss of hundreds of lives, simply because the above possibilities have not happened to occur simultaneously in a high building, filled with people. The probability of this combination occurring, with terrible results, is quite as great as was the probability that the particular combination of circumstances would occur, which did occur in the Iroquois Theater, resulting in over 600 fatalities. The most probable occurrence of an office building disaster would be somewhat as follows: Fire starting during office hours, on the eighth or ninth floor, in a room containing considerable combustible material; no one in the room at the moment; adjacent tenants unable to put out fire when they noticed it; fire department summoned late and unable to control fire on arrival, partly because of no effective interior stand-pipes or other protection; elevator and stair shafts unenclosed, or doors on floor where fire started left open or with cracks admitting smoke; perhaps some elevators making a few trips and then jammed with panic-stricken people all trying to get in at once; stairways likewise jammed, if not cut off entirely; firemen able to rescue a few people by carrying them down fire escapes, or from adjoining roofs, until cut off by flames issuing from windows; persons remaining on upper floors then left without any means of escape, with fire spreading more or less rapidly from floor to floor, either through elevator and stair shafts or through outside or court windows.

"7.—Obviously this hazard increases in direct proportion to the height of the building."

These authorities stress the apparent impossibility of enforcing the regulation of building construction and interior protection and of securing adequate fire-fighting equipment to cope with a catastrophe in a building more

than 10 stories high. If it were possible to secure ideal conditions in every other department, fire-proof buildings might be allowed to go to any feasible height. Failing these ideal conditions, as practically every building does, it must be concluded that, on the score of fire hazard, it would be necessary to limit buildings to not more than 10 stories, or 125 ft.

Congestion on Streets and in Public Conveyances.—Congestion of traffic on country roads and on main thoroughfares outside the central districts will not be discussed in this paper. The problem to be considered is that of determining a sound relation between volume of building and width of adjacent street, so that congestion will not result. Subways are an over-costly and unnatural means of transportation, except where conditions have become so aggravated that they are the only solution and in such case the writer believes that the inflated property values and rentals should be the source of the extra cost of subway construction over that of surface or elevated lines. On this assumption the writer will consider that, in or near the business center, the down-town streets should take care of surface cars, of the vehicular traffic having business there (with short-period parking regulations), and of the pedestrian traffic all without undue danger or inconvenience from crowding.

For study, assume a typical 66-ft. street without car lines, on the theory that such lines have been re-routed to near-by wider, or less crowded, streets. Such a street may be given over to two 15-ft. sidewalks and a 36-ft. pavement. A 15-ft. sidewalk, after deducting for posts, sweeping boxes, and obstructions at show windows and at entrances to buildings, will accommodate two lines of pedestrians in each direction without overcrowding disagreeably. If these pedestrians move at the rate of 3 miles per hour, with a spacing of 9 ft., center to center, the sidewalk will accommodate the passage of about 3 500 per hour in each direction.

The writer has not sufficient data to calculate with any degree of authenticity the pedestrian traffic produced by high buildings, but estimates can be made which may be illuminating.

If a street of office buildings, such as Upper Wall Street, New York, or La Salle Street, Chicago, is considered, the normal occupancy of such buildings might be at the rate of about six persons to a suite of 600 sq. ft., assuming a reception room with one stenographer and one client, and three private offices, two with one person each and one with two persons, including callers or clients. A building fronting along an entire 300-ft. block, 125 ft. deep, to an alley, with 37 500 sq. ft. gross area and 30 000 sq. ft. net rentable area (after deducting corridors, elevator shafts, etc.), would accommodate 50 such suites, or 300 people on each floor. If 80% of these leave the building between 12 noon and 12:30 P. M., or between 5:00 and 5:30 P. M., each floor will deliver to the sidewalk a volume of pedestrian traffic equivalent to 480 people per hour. If one-half the pedestrian traffic on such a street moves north for three blocks, say, to restaurants, a car line, or a subway entrance, the two northbound pedestrian lanes on the sidewalks in the block nearest the car line would receive pedestrians at the rate of $\frac{480 \times 3}{2}$, or 720 per hour.

As the convenient capacity of these lanes is estimated at 3 500 people per hour, the sidewalk could comfortably accommodate the pedestrian traffic from buildings, say, 5 stories high, along the street. This assumes that no pedestrian traffic enters this third block from farther south nor from side streets.

It is obvious that this is the roughest kind of speculation, but it seems to indicate that it is senseless to expect 15-ft. sidewalks to carry conveniently the foot traffic from 16 or 20-story buildings. Much further study is necessary in order to reach any rational factor for limiting the height of buildings on this basis, but it is the writer's opinion that if buildings higher than, say, 10 stories are constructed, additional sidewalk space should be provided by arcading the first story to a depth of 3 ft. for each two additional stories. This should be done for each block, and where a sidewalk carries much through pedestrian traffic in addition to that originating in the immediate vicinity, a deeper arcade must be provided.

The pedestrian traffic on streets which are devoted essentially to retail business or shopping rather than office buildings is more a factor of the character and popularity of the business than of the height of the buildings. Large moving picture theatres, "five and ten cent" stores, and bargain stores produce much greater volumes of pedestrian traffic than banks, markets, or stores selling higher priced commodities. With these also, the writer believes it is feasible to determine by exhaustive study, reasonable regulations for requiring arcades or a set-back at the street line, where either the character of the business or the height of the building make it necessary. In every city which has or is likely to have a population of more than 25 000, such regulations would be reasonable on streets in the main business center which are less than 80 or, perhaps, 100 ft. wide. The city plan should determine which streets need this treatment and what the regulation should be. This is not a direct height limitation, but an alternative to accomplish this phase of the purpose of height limitation.

To estimate the volume or height of buildings for which the typical 36-ft. pavement in the 66-ft. street is adequate, is still more complex than the sidewalk problem. Such a pavement will accommodate a line of parked vehicles on each side (parallel parking) and one line of moving vehicles in each direction. In practically every business or office building center of any importance the idea of providing unlimited parking along the curb has necessarily been abandoned. Such parking must be restricted to an hour, or even less, or prohibited altogether. Limiting the height of buildings does not add materially to the carrying capacity of the lanes next to the curb, for these lanes are used for loading and unloading and are not useful for moving vehicles.

The two flowing lanes will carry, at a speed of 15 miles per hour and a spacing of 50 ft., center to center, about 1 500 vehicles per hour in each direction. With traffic officers closing the cross streets on a one to one schedule, the capacity of these lanes, due more to the delay of starting and stopping than to the actual closed time, is reduced to 500 vehicles per hour.

On any important 66-ft. street it would seem fair to assume that half the carrying capacity of the street should be kept free for through traffic not

destined to or from any point on the street itself, and that the other half of the carrying capacity at any given point might be assigned to the service of the buildings along the street for, say, a distance of five blocks. In other words, a traffic rate of 250 cars per hour for all five blocks, or an average of 50 cars per hour per block, would be the service to be expected from this street.

The traffic service demand of buildings is also practically unknown to the writer. Let us speculate as has been done previously in this paper. On the office building street with 50 suites accommodating 6 people each, at any given time, one person might be expected to come or go from each suite each hour. That would mean 50 people per hour coming or going from each floor. It is doubtful whether more than 1 in 20 or 30 of those entering or leaving an office building during business hours come or go by automobile. If 1 in 30 is assumed, the calculation would permit a 30-story office building on the 66-ft. street.

During the hour from 5:00 to 6:00 P. M., however, people leave the buildings at the rate of about 480 people per floor per hour. If 1 in 30 uses an automobile at this hour, there will be 16 cars per floor, or 160 for a 10-story building, which is three times the quota assigned to each block as its share of the traffic, which will tax the street to capacity.

It is evident that these speculations could be turned into genuine calculations by making studies of the various factors entering into the problem for any given actual street or block. The writer believes it is safe to say, however, that any height limit established on the basis of traffic accommodation in the streets could not permit buildings more than 8 or 10 stories high on 66-ft. streets, nor more than 16 stories on 80-ft. streets. Circumstances such as car tracks, moving picture theatres, the necessity for carrying very large volumes of through traffic, or, conversely, the lack of through traffic and the low traffic demand of certain types of buildings, indicate the necessity for studying the problem for every given situation or for every block.

In general, however, the writer believes that limits could be fairly set on the basis outlined, and that any building exceeding those heights should be required to arrange for arcades or set-backs at the street level for the entire block frontage.

Light and Air.—In support of the necessity for light and air in downtown streets and buildings the writer will quote from "Studies on Building Height Limitations,"* as follows:

"I will talk particularly on the subject of the need of sunlight in relation to colds, coughs, influenza, bronchitis, and pneumonia; and also on the subject of over-crowding on the street cars in relation to the same infections enumerated above.

"First, I want to bring to your attention a series of charts showing the trend of health in Chicago during the last fifty years. Since the best evidence we have as to health is the death rate, these charts show the death rate of the city by years and decades. The first shows an almost incredible improvement. There have been ups and downs, but the unbroken trend has always been sharply downward.

* Prepared for the Chicago Real Estate Board by Dr. W. A. Evans, Health Editor, *Chicago Tribune*, and formerly Health Commissioner of Chicago.

"When we come to analyze this chart more closely—dividing the year into parts—winter, spring, and summer, autumn, or taking January, February, and March as an indication of conditions in winter, and July, August, and September as an indication of conditions in summer, we discover a remarkable fact:

"First, that the improvement in health in the fifty years has been due to improvement in summer health. Winter health is about as bad as it was fifty years ago—whereas the summer death rate has fallen from a maximum of forty to a minimum of ten.

"When we chart out the monthly distribution of deaths by ten-year averages, we find that in the 1870 decade the summer months, July, August, and September, had the high mortalities, whereas in the 1910 decade, the mortality is about the lowest of the year. On the other hand, the January, February, and March mortality has mounted.

"Analyzing further, we find that there has been great improvement in the health as regards typhoid fever, diarrhea in children, and death of babies under one year of age—types of causes largely responsible for the old time high summer mortality. On the other hand, a study of bronchitis, influenza, and pneumonia, shows that those diseases are far worse than they were in the 1870 decade. The death rate from them is not quite as high as it was in the Nineties, but it is much higher than it was in any decade prior thereto.

"This study proves two truths: First, we have controlled every disease against which we have seriously contended. Examples, typhoid fever and the summer complaints of babies.

"Second, in a half century, in which we have almost wiped out typhoid fever, pneumonia and its closely related maladies have become materially worse.

"In all the list of germ-caused diseases, the only group that is not coming under control, in fact, is not practically under control, is the pneumonia group, the acute respiratory infections.

"We know about what to do, in fact, we have more scientific information than we had as to typhoid fever when we decided to act in a large way against that disease.

"Are we willing to approach the question of the height of buildings from that point of view?

"There are several kinds of efforts that must be made to meet the problem of the acute respiratory diseases. To enumerate:

"*A.*—Efforts against the disease directly, such as reporting, hospitalizing, intermediate and terminal disinfection, laboratory diagnosis, vaccines, curative sera, epidermologic procedures.

"*B.*—Efforts to improve vitality, winter sports, vacations, better habits and customs.

"*C.*—Efforts to improve the environment, ventilation, smoke prevention, street cleaning, prevention of spitting, sunlight and air in streets, zoning limitation of height of buildings.

"The limitation of the height of buildings relates itself to the problems of the acute respiratory diseases by reason of the decrease of sunlight, diffused light, and air circulation on the pavements and the atmosphere between the buildings, and by reason of the interference with light and air inside them, and also by reason of the congestion on the street cars during the peak hours.

"The surface of the pavement and the lower stretches of the atmosphere are being constantly polluted. Sunlight and drying are needed to sterilize it. If it is not so sterilized, we suffer infections, principally of the respiratory tract. The sun is less efficacious in the sterilizing process in the winter, because of the short effective day (7 hours of effective disinfection as against 13 for summer), the more pollution of smoke, dust, and cloud—the sharper angle."

Charles B. Ball, M. Am. Soc. C. E., Chief Sanitary Inspector, Chicago, enumerates the effects of dark and poorly ventilated rooms upon the persons who occupy them, as follows:

"These effects may be classified as:

"1.—Physiological:

(*a*) Loss of color;
(*b*) Loss of appetite;
(*c*) Loss of weight;
(*d*) Eye strain and inefficient vision;
(*e*) Nervousness.

"2.—Mental Effects:

(*a*) Depression of spirits;
(*b*) Irritability.

"3.—Moral Effects.

"Perhaps it is not fanciful to inquire whether working in darkness does not affect the spiritual and moral sense. In the Scripture we read (John 3-19) the condemnation of those men who 'loved darkness rather than light because their deeds were evil.' May we not consider the converse true, namely, that the worker in a dark place suffers a distinct lowering of the moral and spiritual sense because of the lack of a natural environment? May it not be true that the irritability which comes to one who feels the supply of light insufficient to convenient working, results in a distinct lowering of moral standards.

"While conclusions of this kind are possibly speculative we may well think about them.

"*Conclusions.*—It appears from the evidence at hand, though not at present susceptible of direct proof, that if tall buildings exceed in height at the building line more than 1½ times the width of the street on which they abut, such buildings, by reason of proximity to each other, reduce light and ventilation at the street surface and in the rooms which they contain to a marked degree.

"No economic argument based upon considerations of land value, rate of return on investment, and the like, is sufficient to justify the hazard to health and general welfare imposed by the construction and occupation of high office buildings to double or treble the safe height of 1½ times the width of the street."

The writer believes it must be conceded that, under ideal conditions, buildings should be limited in height to the width of the street, or at most one and one-half times the width of the street, or for the 80-ft. street, say, 125 ft., or 10 stories.

Spreading the Business Area.—There is some value to a city in spreading the district of highest values over a large area and thus among many owners, as contrasted to a few owners, who tend to control a city's development for the sake of ever increasing the value of their holdings. Furthermore, the argument of the value of extreme concentration for convenience sake is scarcely valid in any but a few exceptional cases. A financial district built up entirely for that use to a height of, say, 10 stories would accommodate, within the compass of a few blocks, all the financial and allied interests of a very large city center. Most business districts are not well specialized. The Chicago Loop District, for example, has a number of 20 and 22-story buildings, but the average height of buildings in the area is only 8 or 9 stories, and a large proportion of the older buildings have not been converted to any of the specialized

uses which find their most advantageous location in the Loop. If this district were entirely occupied by banking, office building, hotel, theater, and retail store uses, for which it is best suited, it could accommodate, under 10 or 12 stories, all the business in these classifications for a city at least twice the present size of Chicago. More or less complete specialization of such an area will give the desired concentration for purposes of convenience in transacting business, without creating any necessity for very tall buildings.

The greater value of spreading out business centers, or even of limiting main centers for the advantage of outlying business districts, lies in the fact that such a tendency toward decentralization of business is likely to lessen the congestion and its corresponding problems, both economic and social. Of course, the design of the transportation system will have more to do with such decentralization than the limitation of building heights.

Architectural Effect.—The writer is among those who a few years ago favored a flat height limitation because of its tendency to produce a uniform sky line, but who have been converted to the possibilities of a sky line such as that being created in New York by the restrictions of the zoning ordinance height regulations. In districts of lesser heights, the desirability of a fairly uniform height along a street is unquestioned, and height limitations justify themselves for this result alone. In the districts of great height, however, that height limitation seems best, from the architectural standpoint, which permits the greatest height but which tends to produce uniformity in the cornice line of the main body of the buildings and, in turn, makes possible a variety of tower and set-back effects. The ultimate result of this type of regulation is not yet evident, but now no great appeal can be made for a low flat height limit for architectural reasons.

Among the advantages of high buildings as mentioned previously, concentration of business on small areas has been discussed. For the point that the high buildings are sanitary and well lighted in themselves it may simply be said that the 8 or 10-story building can be and now usually is as well constructed and equipped as the 30-story building. In the monumental treatment of buildings, unlimited height is advantageous. Height limitations of the type in effect in New York are still more advantageous.

The rock on which nearly every height limitation project has been wrecked is that high buildings earn returns on very high land values. The owners of business property of high value appreciate the arguments for height limitations, but when such limitations threaten to emperil their valuations, they, like all good business men, balk. Consider how well American height limitations meet the ideal requirements (ideal only in the sense that they are the actual human requirements for sound public policy) and then re-examine this critical matter of the return on high priced land.

It has been found that the question of ideal height limits requires much more study in order to reach sound and rational factors for setting the limits in any block or for any street. Can it be safely said, however, that the considerations of fire hazard, street congestion, light and air, and decentralization of crowded centers, tend toward a reasonable maximum limit of 10 stories

or, say, a volume of ten times the area of a given lot. The heights allowed by various zoning regulations are, as follows:

City.	Maximum height district, in feet.
Atlanta, Ga.	150
Chicago, Ill.	264
Hoboken, N. J.	185
Milwaukee, Wis.	125
Omaha, Nebr.	175
St. Paul, Minn.	130
Washington, D. C.	110
Akron, Ohio	136
Newark, N. J.	150
St. Louis, Mo.	150
New York, N. Y.	250

These regulations carry various modifications with respect to towers and additional height with set-back, but with these, the largest cities with zoning ordinances in effect, only three—Milwaukee, St. Paul, and Washington—have regulations which approach the figure which will realize the purposes for which the regulations are intended.

The force behind the high-building tendency in these regulations is largely that of the owners of property on which such buildings may be constructed. Their demands must be met. Until public ownership of land can be realized, or perhaps some single-tax system (and, in fact, the full success of city planning in general cannot be reached until that time), it is reasonable to expect owners to demand the opportunity for a fair return on the market value of the land.

There are, however, certain factors in this phase of the problem which are often overlooked. One factor of great importance is that with a limit so high as to be virtually no limit, as in Chicago with 264 ft., the opportunity to improve property with very high buildings is less than it would be by a lower limit. For example, the central district in Chicago is now built up to an average height of 8 or 9 stories. Streets and transit facilities are already so badly overtaxed that business is being forced from the Loop area. No arrangement or use of the streets on, above, or below the ground, will make it possible to accommodate successfully an average height of more than 12 stories over the whole area. In other words, the owners of property who are able to improve soon with buildings 20 stories high (the present limit), may succeed in earning a return on the "20-story value" of the land. For every property improved to 20 stories, however, some other property will find the business demand so taken up that it can rent only 4 stories, say, and the value of the land will decline accordingly. A 12-story limit in this district besides achieving important public good in relieving traffic congestion, improving public health, etc., would permit the reaping of the full useful value of the area. This value, however, would go to the pockets of all the property owners, several thousand in this case, instead of into the pockets of only those who are able to improve ahead of the others.

Another factor is the decreasing return on buildings above a certain height, which varies with different conditions. Mr. George C. Nimmons has shown the economic fallacy of the very high building. Due to the area occupied by elevator shafts, supporting members and foundation, the actual rate of return normally declines for buildings of more than 20 stories, and the 20-story building has little advantage over the 12-story building, and none at all if it cannot all be rented, if it is "overbuilt" to any degree.

Assume that the hope for big reward is so strong in every one of the property owners in the central district that they insist on a height limit greater than the public good would dictate, what can be done to improve height regulations?

The suggestions made seem to be worthy of consideration in this connection and as a conclusion to this paper:

First.—Based on intensive studies in each case, buildings seeking to go above a fixed height, say 10 stories, should be required to arrange for a widening of the street by arcades or by setting the buildings back. If necessary, this may be required to be arranged for a whole block before any building is permitted to exceed the fixed limit, as an arcade along the front of one building alone might not be very useful. This, of course, is not a new idea, but it is offered to tie together the matter of high buildings and wider streets and to place the burden for widening the streets where it belongs, namely, on those who profit most by them.

Second.—Any building seeking to go above the fixed limit of, say, 10 stories, should be positively required to put in the most nearly adequate fire protection and exit appliances that it is possible to devise.

Third.—If the streets and the lower floors of buildings are to be made disease-breeding canyons, other means must be devised for controlling the respiratory diseases, eliminating eye strain and nerve strain, living in an unnatural environment cut off from sunlight and fresh air. It is not likely that these means can be devised, but engineers will be searching for them while more high buildings are being constructed and until the evil results become too serious to endure. Then we will begin tearing down and rebuilding our cities. This will be expensive, but by that time we will know better how to build well for human needs.

HOUSING DENSITY REGULATION

By Robert Whitten,* Esq.

In a comprehensive plan of zoning, the regulation of the area of the lot with relation to the number of families that may be housed upon it, performs a very important primary function in securing an appropriate distribution of population and, at the same time, serves other important zoning purposes and supplements other zoning regulations.

The use of the family as a unit of density regulation is open to the objection that the size of a family is variable. There are families of all sizes. A single individual keeping house by himself is considered a family, and there may be families of as many as twelve persons. According to census statistics the average family contains from 3 to 5 persons. In 1920 the average size of a family in the urban communities of the United States was 4.2 persons. For the 68 cities of more than 100 000, the average family ranged between 3.6 and 4.9 persons. For any considerable residential area, the population density will vary directly with the number of families housed therein. The law of averages will take care of the exceptionally large and of the exceptionally small families. For all practical purposes the human load on the land can be limited effectively by regulating the number of housekeeping units.

There are some difficulties in the rigid enforcement of the area per family requirement. A house or apartment may be subdivided for light housekeeping, and unless structural alterations are made, requiring the issuance of a building permit, the strict letter of the regulations may be violated. The same difficulty is involved in a rigid enforcement of all tenement house regulations. The housing code regulations governing the erection of tenement houses usually apply to all buildings arranged for more than two families. No large proportion of the population of a city will be housed in light housekeeping, makeshift apartments. With minor exceptions the regulation of the construction of complete housekeeping units will effectively control density. Practical experience under family density regulations both in this country and in England proves that the opportunity for occasional evasion does not materially interfere with their effectiveness in regulating density of population.

In some zoning ordinances the area per family requirement is stated in terms of a limitation of the number of families per acre. This, as here used, is precisely the same as requiring a specified number of square feet of lot area per family. Thus, the Paterson, N. J., ordinance provides that in D residence zones and in all business and industrial zones: "No dwelling or tenement house shall hereafter be erected or altered to accommodate or make provisions for more than 110 families on any acre of land or more than a proportional number of families on a fractional part of an acre of land." This is the same as a requirement of 396 sq. ft. of lot area per family, but is a rather involved way of expressing it.

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The area per family requirements must be fixed of course with reference to the prevailing lot sizes and housing types in each community. The prevailing width and depth of lots are usually even more important than the prevailing housing types. If, as in many cities, the normal lot is 40 by 120 ft., giving an area of 4 800 sq. ft., a good unit is provided on which to base the requirements for all the area districts. In the sections most appropriate for single-family houses 4 800 sq. ft. of lot area may be required. In those sections where the two-family house is desired, a requirement of 2 400 sq. ft. per family will be appropriate. In an area where small apartment houses should be permitted, a requirement of 1 200 sq. ft. per family will permit a four-family house on a lot of the same size as that required for a two-family house in the 2 400-sq. ft. district, or for a one-family house in a 4 800-sq. ft. district. On a proportionately larger lot, an apartment for any number of families could be constructed. In the higher-value sections, in which large high-class apartments would be the appropriate development, a requirement of 600 or 400 sq. ft. per family may be provided. For cities in which the normal lot depth is 100 ft., a scale of 4 000, 2 000, 1 000, and 500 sq. ft. is usually more appropriate than the scale based on the 4 800-sq. ft. area. In many cities a variation in any such scale will be required to meet the conditions created by special types of housing or lot sizes.

In any large city there are a few limited areas where the placing of a family density regulation is unwise. This includes, in general, the apartment hotel section. This is a section where location and high land values make the possibility of congested tenement conditions very remote. The apartments, as a rule, will be occupied by small families of adults. There is no problem of finding play space for great numbers of children. The high-class hotel or kitchenette apartment needs no regulation as to family density, provided it is located in a limited high value district where it will not spoil adjacent dwelling-house areas and where there is no danger of the development of congested tenement conditions. Examples of zoning ordinances in which limited apartment-hotel sections are exempt from the general area per family requirement are those of Atlanta, Ga., Indianapolis, Ind., Providence, R. I., and Columbus, Ohio.

In private dwelling sections in suburban areas, where it is desirable to promote or protect a more open type of development on larger lots, 7 500 or even 9 000 sq. ft. per family may be a proper requirement. In the Indianapolis ordinance, 7 500 sq. ft. is required for such an area; in Cranston, R. I., 8 000 sq. ft.; and in West Hartford, Conn., 9 000 sq. ft. A section of large lots and beautiful homes is always a fruitful field for exploitation by those who see an opportunity for gain by dividing existing lots or crowding more families on the same lot area. The character and charm of the residence section may thereby be destroyed. Unless each builder is required to make a proportionate contribution to the light, air, and open spaces of the neighborhood, there will be a constant trend toward more intensive building.

In jurisdictions where there is doubt as to the legality of zoning regulations establishing single-family, one or two-family, and apartment-house dis-

tricts, the same result in some cases may be secured by a combination of area per family, lot width, yard, and percentage of occupancy regulations. The West Hartford zoning regulations do not exclude two-family houses or apartment houses from any district, but the area per family requirements combined with the lot width and side yard requirements, make apartment houses uneconomic in the areas designed for one or two-family houses.

The area per family requirement should be applied to the business and industrial districts as well as to the residence districts. No greater housing density should be allowed in an industrial district than in an adjacent residential district. Unless this is done there will be a premium placed on the erection of tenements in the industrial areas. Congested slum conditions may thereby be created. In some instances it will be desirable to place an industrial district in the area district requiring the largest lot areas per family. In most cases a requirement of 4 800 sq. ft. per family in a heavy industrial district will be the practical equivalent of the exclusion of dwellings from such district. Although this accomplishes the same purpose as direct exclusion, in securing a segregation of dwellings from industry, it does so in a less rigid way and in a manner less open to legal criticism.

The area per family requirement should be supplemented of course by the usual height, percentage of lot, and yard regulations. Minimum side, rear, and front yards should be required for all buildings erected in residence districts. Height and percentage of lot occupancy regulations are usually desirable. Bulk regulations, however, no matter how carefully drawn, cannot be as effective in regulating density of population as required-lot-area-per-family. There is always a tendency to crowd more families into a given space. Rooms are made smaller and the number of rooms per apartment reduced. Bulk regulations cannot prevent this. Crowded tenement conditions are likely to result.

For a residence section occupied by people of a given racial, social, and economic class, there is a certain normal rent per house or per apartment. Land values are based on the net return from the rentals received from the customary improvement of the lot. In a given district, the land values increase with the number of families normally housed on a lot. The speculative builder sees a large opportunity for gain in crowding more families on a lot than has been customary in the neighborhood. He buys a vacant lot at a value based on its net return from use for a one or two-family house and builds a four-family house, and is usually able to pocket the difference between the cost of a single lot and the cost of two lots. After a considerable number of such small tenements have been erected, and land values have thereby been practically doubled, the same process is repeated. The speculative builder discovers a profit in building six or eight-family tenements on lots of the size originally used for one or two-family houses. This process of a more and more intensive use of the land and a corresponding increase in land values continues until at length slum conditions are produced.

As long as there is a premium on more and more intensive housing, that is the type of housing that will be most largely supplied. This is particularly

true of the cheap rental type of housing. The tenant has to take what is supplied. In a growing city, he is usually required to pay the same or a somewhat higher rent for ever smaller and smaller quarters in an ever more and more congested and undesirable neighborhood.

Under present conditions in limited sections of most large cities there is a concentration of jobs for cheap labor, that furnishes economic if not social justification for crowding in the near-by residence areas. Where such conditions exist, regulations will be practically forced to recognize them. An allowance of one family for each 1 000 sq. ft. of the area of the lot will adequately satisfy this housing demand under most conditions of industrial concentration. It would result in a probable density of 40 000 to 60 000 per sq. mile. This density although undesirable is low as compared with existing densities in New York and existing trends in limited areas in many other cities. Many of the 4 and 5-story tenements now being erected in New York have only 200 sq. ft. of lot area per family. This creates conditions of tenement congestion from four to five times as severe as the suggested minimum requirement of 1 000 sq. ft.

If in a cheap tenement section, as just described, land values are already based on the housing of eight families on a 40 by 100-ft. lot, it is too late to apply the 1 000-sq. ft. requirement. In zoning it is not practical to adopt regulations that will reduce land values generally in any considerable area. The pyramiding of land values and congested housing, however, can be stopped where it is. Family density regulations are necessary to prevent the eight-family house from increasing to sixteen and the sixteen to twenty-four.

It is assumed that with the housing density regulations, supplemental zoning regulations and thoroughfare and transportation facilities will be provided, that will bring about a corresponding distribution of business and industry. Concentration of industry in one part of the city tends to a congestion of population in the adjacent sections. Similarly, the congestion of population in any part of the city tends to a concentration of industry in the adjacent sections. The best conditions both for the industries and for the housing of the workers can be obtained where there is a proper balance between the areas devoted to industrial use and the adjacent areas available for housing.

Zoning regulations supplemented by thoroughfare, transit, and transportation planning, can promote the decentralization of business and industry to sub-centers or to suburban or satellite towns. With such appropriate decentralization there is no economic justification for the housing of the mass of the population in tenement houses. Abundant land will be available for housing convenient to the various business and industrial centers.

One fundamental means of traffic relief in the great city is to take from the traffic or business center the industries and businesses that do not properly belong there; and especially all those that can be more economically conducted in sub-centers of business and industry located throughout the city and its suburbs. Ideally, the big city proper should be devoted primarily to commerce and to the homes of the people engaged in such commerce. The

suburban towns and satellite cities clustering about the big commercial city should be devoted chiefly to industries and to the homes of the workers employed in such industries. This arrangement would reduce the traffic, transit, and housing problems to a minimum and increase commercial and industrial efficiency.

The prevention of the increase and spread of tenement congestion in the housing areas near the business center of the great city is one of the ways by which the decentralization of industry can be most effectively promoted. Housing congestion leads to increased industrial concentration and every increase in industrial concentration leads to increased housing congestion. Just which is cause and which is effect is often hard to determine. The one thing that is clear is that congestion breeds congestion. A zoning regulation requiring a specified number of square feet of lot area for each family for which a tenement is arranged, is the most effective remedy yet devised to cope with this congestion problem.

Housing density regulation has played a prominent part in England in applying town plans to the unbuilt areas. The density requirement is usually stated in number of houses per gross acre. The permitted per acre densities range from about four single-family dwellings to a maximum of twenty single-family dwellings. The town plan divides the housing areas into zones with a varying family or building density requirement. The regulation applies to tracts or land units and not, in the first instance, to particular building plots. The owner or subdivider of a land unit may provide for various types of houses and lot sizes as long as the prescribed housing density is not exceeded for the area as a whole, including the area devoted to streets, parks, and open spaces. This power to control family density is one of the most useful and important features of English town-planning practice. It will be equally important in the United States when a way has been found to apply comprehensive planning to unbuilt and unsubdivided areas.

However, density regulations should not be applied to large unsubdivided areas so as to prevent an appropriate variety in housing types and lot sizes. Although the single-family detached house is a favorite type in all American cities, there will be few large residential neighborhoods in which a limited number of two-family houses, group houses, or apartment houses will not be economically and socially desirable. Although Forest Hills Gardens, New York, is an attractive residence suburb, primarily for single-family detached houses, it has small areas for apartment houses, and other limited sections for group houses, all clustered about its carefully designed and controlled community and store center. Zoning should not bar the possibility of such layouts in any large unsubdivided area.

To accomplish this, the area per family requirement in the unbuilt areas should be based on gross areas as applied to each particular subdivision. Thus, a gross area requirement of 8 000 sq. ft. per family would be reduced to a net area requirement of 4 800 sq. ft. if 40% of the gross area were devoted to streets, parkways, or small parks. This would serve the purpose of balancing the actual density of population and the contribution of each owner

of land to the open development of the entire area, and, at the same time, make it easier for the individual subdivider to appreciate the advantage of contributing the street, parkway, and small park areas required under the planning regulations. In a general way, the maximum number of dwellings to be erected on each tract would be fixed and the only question as between the subdivider and the planning authority would be as to how a small portion of the open space required should be divided. Whether the size of the individual house lots should be reduced and the area thus saved used for wider streets and small park reservations; or whether the maximum yard area should be retained in the individual house lots? To encourage more adequate provision of park area, it would probably be desirable to allow a more than proportionate deduction from lot area for area devoted to parks. It might be provided, as in the recent West Hartford zoning ordinance, that area devoted to small parks or parkway strips may be given double value in computing gross area for the purpose of determining the number of families that may be provided for within a subdivision.

In American zoning ordinances the density regulation, whether families per acre or area per family, applies to the net area of the particular lot on which the proposed building is to be placed. As American zoning ordinances are applied for the most part to already built up or subdivided areas, it is essential that the area requirement should relate to each individual lot and building. As in many American cities there are large unsubdivided areas within the city limits and, therefore, controlled by zoning, it is desirable to devise some more elastic means of density control, such as that used under the English Town Planning Act and applied in modified form in the West Hartford zoning ordinance.

The chief purposes of family density regulations may be summarized, as follows:

1.—The prevention of the development or spread of areas of excessive tenement house congestion such as those that exist in most large cities and in many smaller ones.

2.—An escape from the vicious circle of more intensive housing resulting in higher land values and of such higher land values resulting, in turn, in greater housing congestion.

3.—An escape from the vicious circle of congested housing causing a concentration of industry in near-by areas and of such concentration of industry resulting in increased housing congestion.

4.—The fixing of the most desirable standards of density for housing development in the unsubdivided areas some years in advance of the time when it may become profitable to a few developers to use lower standards.

5.—The grading down of the concentration of the residential population in accordance with economic and social requirements from the high value center or centers outward.

6.—The securing of an open, detached-house type of development in appropriate areas.

7.—The indirect control of multi-family house construction by area per family requirements that make the construction of an apartment house or of a two-family house uneconomic in all but very exceptional cases.

8.—The possibility of accurate estimates of future population and densities in the various residence areas as a basis for a more accurate forecasting and planning of future needs for all kinds of utilities and public and municipal facilities and services.

BUILDING LINE ZONING

By Lawrence V. Sheridan,* M. Am. Soc. C. E., and J. Clyde Hoffman,† Esq.

For many years, the establishment of building lines has been a subject of consideration by those who have dealt with the problem of the orderly development of growing cities. This study has been prompted by a desire to promote proper living conditions as well as to provide for the future street requirement of the city by arranging for a uniform distance back from the street line, in front of which buildings should not be erected. The open space adjacent to the street which is so necessary for light, air, and the general well-being in residence sections is thereby secured, and opportunity is afforded for landscape treatment and otherwise adding to the beauty of the home and neighborhood. This has been the real purpose of a building line in most cases where residence districts were involved. Where, however, in the growth of the city, it has become evident that the original width of the street is no longer adequate for the increased traffic, and that it is desirable and necessary to widen the street at a future time, building lines have been established to forestall the erection of buildings over the strip of ground adjacent to the street that may be necessary for the proposed widening.

Most of the cases considered by the Courts in the various States have been of the latter type. It will be seen that the establishment of a building line for such a purpose is the taking of private property for a public use, or at least a partial taking at the time the restriction is placed, to be completed when the street is actually widened.

Building lines for such purposes usually involve property of high value, and in many instances are the cause of litigation. This has resulted in a line of decisions setting forth the doctrine that the establishment of a building line is, in effect, the taking of private property, and to be valid, compensation therefor must be made. The Courts have been reluctant to consider the fixing of the building line as coming under the police power; and the validity of any attempt so to do has been a serious question. Therefore, the usual practice is to establish a building line by the exercise of some form of eminent domain.

In more recent years, since city planning in its present form has been adopted throughout the country, one of the first things to engage the attention of those interested in such work, is the building line. It is necessary properly to plan for the development of new areas, and just as necessary to plan for the making over or reconstructing of the older and built-up parts of the city.

If all such building lines must be established by the expensive method of condemnation, the cost would be too great to permit of it being done uniformly and at a time to forestall the erection of buildings, the removal of which might be required later at great expense to the city. To avoid this difficulty, in new and undeveloped areas, the Board that approves plats and subdivisions

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may be empowered to require the designation of building lines on all lots included within the plat, with a penalty or forfeiture clause included and with the right vested in the city or any lot owner to enforce the penalty or to prevent the violation of the restriction. This is the ideal building line. It is a condition or restriction running with the title to the land, and is established without cost, as the subdivider, in most instances, recognizes the advantage to himself in selling lots with such safeguards to the new owners. Such lines have been established, and serious damage has resulted in some instances. There is a lot in Indianapolis, Ind., occupied by a fine old residence built many years ago, which is now used for offices, and, it is said, at a loss to the owner. This lot is near the section of the city now used almost entirely for office purposes, yet it is subject to a building line established before the house was built and when the neighborhood was entirely residential. It is impossible now to improve it with a business building of the character which would be necessary to bring in an adequate return. The damage, of course, is due to the fact that the building line was established as a permanent restriction on the land. Generally, plats should provide that the building-line restriction shall be in effect for a certain period of years, twenty-five years being a reasonable term in most instances.

The City Plan Commission of Indianapolis requires that building lines be included as parts of all plats which are submitted to it for approval, and all plats must be so approved before they are eligible for record. It has been found that the establishment of such lines tends to increase the value of property because of the protection against unthinking or unscrupulous builders who would build out beyond the line to obtain a special advantage for themselves.

To take care of the conditions which may exist in older and built-up business or semi-business parts of a city, when it is deemed necessary to prepare for future widening of streets, various procedures, in some form or other of condemnation, have been devised, usually providing for the postponement of the payment of damages until a future time fixed, or dependent, on the removal of the existing buildings or the actual taking of the ground. These methods have been more or less satisfactory, but they are carried out at great cost to the city.

The Indianapolis Plan Commission was confronted with this important and perplexing problem. It proceeded on the theory that the establishment of a building line over built-up property where the sole purpose was to prepare for a future widening of the street was in reality a part of such widening, and not such a regulation as would fall within the police-power doctrine—that consequent damages must be paid—and, therefore, not within the province of zoning.

However, following the existing law in Indiana with reference to the payment of damages and the assessment of benefits in condemnation for other purposes, the Commission submitted a bill to the General Assembly in 1923, which was enacted, granting to the Board of Public Works the power to establish building lines by condemnation and the payment of damages therefor, with certain provisions for the postponement of such payment, and also pro-

viding for the assessment of benefits against the property abutting on the streets along which such building line is established. The law also further provides that the benefits so assessed may equal the damages. It will be seen at once that the cost to the city by such procedure is greatly reduced. In one instance, a 75-ft. building line along 2 miles of Meridian Street was established at a total cost of $5 400. This was for damages to one property in excess of benefits. On all other lots for the entire 2 miles, the benefits and damages were fixed at equal amounts.

Under this law, the right acquired and paid for is merely to prevent the erection of buildings between the building line and the street line. All other rights of ownership remain with the owner until the street is actually widened. The great saving to the city is secured by the reduced cost of the later widening, when it is not necessary to remove expensive improvements. Under this method existing buildings are permitted to stand until the actual widening. New buildings are erected back of the building line. In most instances, owners cheerfully assent to the procedure in order to obtain the advantage accruing to their property by the improvement occasioned by the street widening.

Neither of the types of building lines mentioned comes strictly under the head of "building line zoning", but a reference to the methods therein used, serves well to aid in a clear understanding of the theory of building-line restrictions in zoning ordinances, as well as to offer suggestions as to procedure, etc.

In zoning ordinances, the matter is handled in an entirely different way and with a purpose quite different from the purposes outlined under the condemnation methods. Zoning does not attempt to establish building lines for purposes other than those for which zoning itself is established, that is, for health, safety, or public welfare.

Strictly speaking, therefore, "building line zoning", is not the proper terminology. It would be more apt to say "front yard zoning". In many zoning ordinances the term, "set-back line", is used, but because of the uncertain attitude of the Courts on the question, it is believed to be safer to use the words, "front yard", and as far as possible to adhere to that idea, the same results being obtained as if so-called building lines were established.

Since so-called zoning has been used, and, in fact, long before its use, when the only protection of building was through housing laws, it was recognized that the establishment of side yards in residential areas was a real necessity to safeguard the health, safety, and welfare of the public. From the standpoint of fire protection there is no question but that any measure which would compel owners to provide considerable space between buildings would be of value in event of a conflagration. There are instances in every city where houses are built so closely together that the eaves overlap. A fire in one building is almost certain to be communicated to the neighboring houses. With the great preponderance of frame construction in American cities, it was necessary to secure more effective fire breaks between buildings.

Buildings constructed very close together of course occasion shaded windows. Such houses are unquestionably darkened, damp, and do not contribute to the health of their occupants. Many of the housing laws provide for a side

yard of 4 ft., meaning 8 ft. between dwellings, and zoning ordinances, following the lead of the housing laws, have strengthened side-yard requirements.

Housing laws and zoning ordinances have usually provided for rear yards for much the same reason as side yards. Such requirements have also aided in reducing the percentages of lots occupied by buildings and have thus reduced congestion in dwelling-house neighborhoods.

The arguments for front yards are based on much the same considerations as those for side and rear yards. By their establishment, more light is obtained in streets; there is less fire hazard when the houses are far apart, and, in addition, a greater measure of safety is added on account of the larger yard areas, allowing spaces for children to play unendangered by street traffic. There is no question but that well-defined lines on a street contribute to the happiness and health of all its residents. This contribution is a matter of general welfare.

Many incidental benefits arise from the establishment of such front yards. It prevents an owner from placing his building beyond the other structures and thereby decreasing the value of neighborhood property merely to gain an advantage for himself. This, also, is a matter of public or general welfare.

In many cases the establishment of such front yards aids materially in the widening of streets. It should be borne in mind, however, that street widening should not be one of the fundamental purposes in the establishment of front-yard lines in a zoning ordinance. They are based solely on the considerations of health, safety, or general welfare, and should be sustained because they are a proper exercise of the police power. For this reason it is not customary to establish front-yard lines in other than dwelling-house and apartment-house districts. The same arguments, based on the health, safety, and public welfare, might not apply so well to a business or industrial district, and thereby cause the Courts to look with disfavor on the whole subject.

There has been an effort on the part of some to endeavor, through legislative enactment, to place the establishment of building lines clearly under the police power; and it is quite likely that, with the trend of conditions and the necessity for the remedy, the Courts may uphold such enactments and eventually declare the establishment of a building line for any of the purposes discussed, to be a proper exercise of the police power. Until that time, it will be the wise course to recognize the "front yard" as creating the only building line which should be attempted in a zoning ordinance.

Incidentally, the establishment of front-yard lines contributes to the esthetic value of the city, but as happens frequently in zoning and in efficient city planning, beauty results, not as the first consideration, but as a by-product of intelligent planning.

DISCUSSION

W. W. Crosby,* M. Am. Soc. C. E. (by letter).—The splendid paper of Mr. Knowles furnishes inspiration as well as fact that is of interest to all, particularly to engineers. Those of the latter who cherish the advancement of the profession can but admire the breadth and temper of his presentation and will receive from it encouragement for their further efforts. Even though engineers are now united for zoning, the education of other professional groups and of the "man in the street" is, as Mr. Knowles points out, fundamental for progress. The responsibility for this educational work must rest primarily on engineers, and such papers as this undoubtedly help the cause substantially.

Zoning is a form of co-operation. Healthful co-operation depends on the proper recognition of the community of interest. By intelligent presentation to the Society (as in this paper) and by comment on it by those so well qualified as its members, interest and education in the matter should glow and spread. "On the anvil of Discussion are the sparks of Truth struck out." It is to be hoped they may fly far enough outside to do good.

Mr. Knowles suggests the need for the cultivation of support by the Courts for zoning principles and practice; he is quite right. Possibly a judicial recognition of nuisances other than those to the nose and ear will be thus induced to the great benefit of all.

He exposes briefly a rich "lead" if not an actual "mother-lode" in his remarks under "Skyscrapers".† The menace to the individual, singly and in incoherent groups, from this truly American development of a Frankenstein conception is not appreciated sufficiently until viewed both through a glass crystallized from a strong solution of community interest and in the light of zoning accomplishment. Then the menace becomes plain enough to inspire proper safeguards. At the same time such reasonable uses of the monster should be allowed as will be beneficial to the majority.

Zoning is a practical recognition and application of the principles of democracy on which the United States is based. Democracy means community interest and the good of the majority—not abstract ultimate liberty for the individual and his personal desires or whims irrespective of those of his neighbors and fellow citizens. Belief in American principles must mean belief in and support of zoning; in the right of communities to zone; in the establishment of proper principles and practice for zoning; eventually these must result in a majority recognition of the benefits to be derived by both the public and the individual—either directly or indirectly—from zoning.

Arthur S. Tuttle,‡ M. Am. Soc. C. E. (by letter).—Mr. Knowles has brought out the basic principles on which zoning regulations must rest in order to stand the tests of public endorsement and litigation.

* Cons. Engr.; Location Engr., State Dept. of Highways, Harrisburg, Pa.

† See p. 636.

‡ Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

The municipal authorities of New York took a pioneer step in 1916 when they adopted the first comprehensive zoning plan. In no other city are property values so high. The fact that the Act has been sustained by the Courts in every particular is conclusive evidence of the soundness of the principles used. At that particular time, real estate was inactive, and values, in many sections, were seriously menaced by reason of the helter-skelter methods used by individual builders in developing their holdings to suit their own particular needs.

The zoning principle was welcomed by nearly every interest as the only means of restoring stable values, notwithstanding that in many instances its application to particular holdings often carried with it a temporary depreciation in value more or less serious in extent. Only three districts were recognized for use restrictions—residence, business, and unrestricted; five districts for height—1, 1¼, 1½, 2, and 2½ times the width of the street on which a building has frontage, with 50 ft. considered as the minimum width and 100 ft. as the maximum; and five districts for area—designated A, B, C, D, and E.

Since the adoption of the plan, and more particularly during the active period of building following the close of the World War, the discussion of zoning in New York has been continuous, but it has been confined almost wholly to particular districts where changes have been sought. As stated by Mr. Knowles, those which have been favored in the majority of instances are in the nature of increases rather than decreases in the restrictions.

The resolution of 1916 has been subjected to only a few amendments, and most of these are of a minor character, the only exceptions of note consisting of adding three new height districts, namely, a ¼-times district, a ½-times district, and a ¾-times district, to meet special conditions where exceptional treatment was deemed essential, of imposing a 10-ft. set-back in the E-District, and of adding an F-District designed to insure detached house development, coupled with the imposition of a 15-ft. set-back.

Under the law, the Board of Estimate and Apportionment is charged with responsibility for setting up and amending the zoning resolution and for applying it to areas of substantial size. Cases relating to the treatment of small parcels are left to a Board of Appeals. The Board of Estimate and Apportionment must give a public hearing, after due notice, concerning a proposed change either in the rules or in the plan, and in no case may action be taken until the merits of a proposed change have been investigated and reported upon by its Chief Engineer and given independent consideration by its Committee on City Plan. That the original plan has been so closely adhered to and that it has been sustained in every particular by the Courts is a strong tribute to the work done by the Committee in charge of the development of the plan as well as to the conservative method which has since been invariably observed by the municipal officers responsible for its maintenance and enforcement, and this notwithstanding the strong efforts that have been made to secure changes which would have introduced additional

restrictions of considerable merit, but of a questionable character from a legal standpoint.

The allowance of a private garage for five cars in a residential district has occasioned much discussion, but the objections have been met in some measure by enforcing the provision against rental of any surplus space.

In the zoning of residential streets it has usually been the practice to exclude the frontage for a distance of 100 ft. from the intersection with business streets, with the expectation that the business development would front upon the latter. Repeated violations of this implied condition, with a prejudicial effect on high-grade residential property, has resulted in the serious consideration of a comprehensive re-study of all such areas to the end that a remedy may be found either through an amendment of the rules or through re-zoning. It will be recognized, of course, that property values on a business corner might be seriously affected if the lot subdivision were such that frontage could only be obtained on the business street and an attempt were made to enforce a residential use.

Before zoning was introduced, the history of the older streets which had grown up as residential, showed that as buildings of a substantial character became old-fashioned or out of keeping with the vicinity, it was the custom to convert them into other and more profitable uses, pending the time of demolition, and thus to secure the maximum revenue they might yield. Stores and industries were thus introduced into what were once exclusive residential areas, and, once introduced, the effect was to speed a similar change in character throughout the section. Since the introduction of zoning in New York, decay of this sort has been checked, and property owners are forced either to renovate or reconstruct such buildings unless an actual change in zoning is deemed meritorious. There is no question but that when the time for demolition has not clearly arrived and the buildings are out of keeping with what is required by the tenants, a serious loss is faced by the owner; but, on the other hand, if the zoning is not respected as reasonably permanent, it becomes only a question of time when all high-grade property must give way to less restricted uses.

The symbols on the plans as originally adopted were placed within the street lines, and their interpretation in many cases could only be made by an expert. In preparing a new edition of the maps, which has been approved, advantage has been taken of the opportunity to substitute new symbols, placed within the block lines, showing clearly the exact boundary of the various districts—a method possessing marked advantage over the one earlier in use and conforming with that generally observed elsewhere.

If any one can doubt the merits of a proper zoning plan, it is only necessary to point out the amazing record of its acceptance throughout the United States as noted by Mr. Knowles. If kept within limits designed to recognize the necessity for light and air, and the general welfare of the community, and if considered along broad lines and within common-sense limits, there is now no question but that zoning can be legally sustained, and any tendency to introduce questionable or extreme requirements should be deprecated.

CHARLES WELLFORD LEAVITT,* AND MAX WAKEMAN WEIR,† MEMBERS, AM. SOC. C. E. (by letter).—Mr. Knowles' paper is clear and concise. Any further discussion compels a more or less re-statement of the matters already so ably set forth.

The right of eminent domain of the sovereign political subdivision has been invoked successfully for public improvements for many years. Its necessity is well recognized. It is exercised to secure to the public such lands as the public requires for parks, transportation, schools, streets, roads, public buildings, etc. Similarly, the police power has been exercised to secure to humanity, safety, health, light, and air.

These two well known and thoroughly acknowledged public rights or powers must not be confused or transposed. They have been established for two separate and distinct purposes: First, to obtain land for public use; and, second, to obtain public control over private activities to the limit of public safety and health.

To illustrate: A zoning ordinance with restrictions as to front-yard depth or set-back lines, may be drawn for some particular built-up community suffering from the traffic ills occasioned by narrow streets. The purpose is obvious. This method of securing the additional street width by invoking the police power is a confusion of the right of eminent domain—it seeks to deprive the owner of the proper use of a part of his land without just compensation. The Courts say "No, this shall not be done."

To illustrate how set-back restrictions can be applied under the police power, let a new development be laid out into streets and building lots and the lots sold with restrictions as to the set-back covering the whole development. The Courts say this may be done because all are participants and the available lot area is known.

In the first illustration, the owner would be damaged because his lot area would be reduced, while his neighbor on a wider street would lose nothing. Further, the loss sustained by the damaged party would go to enrich the public domain. Engineers interested in zoning will do themselves and their clients a great injustice unless they keep in mind a clear conception of the distinction between these two different rights of the public.

In preparing a zoning ordinance and map, the comparatively common error of trespassing on the field of private restrictions must be avoided if success is sought. It must be kept in mind constantly that the police power can be invoked only on the grounds of safety, health, light, and air. The case of Nutley, N. J., illustrates this point very clearly. This case involved the right of use and came about through the opening of a retail store in a residence district. The decision of the Court was that there was no question of safety, health, light, and air involved and, therefore, such zoning against the use of property for a store was an improper invocation of the police power.

The welfare of the whole community must be the paramount consideration and no zoning should be attempted which, in any way, has for its object

* Civ. and Landscape Engr. (Charles Wellford Leavitt & Son), New York, N. Y.
† Cons. Engr., New York, N. Y.

the special protection of any (so-called) class of people from the activities of another class. It must be remembered that the Courts recognize no classification of persons, but look on all as equal in the light of the statutes.

The education of the public and of the public official must be accomplished before great progress in zoning and in the administration of zoning ordinances can be expected. This education must be carried to the people interested, not in technical language such as engineers use one to another, but in language and thoughts that the people understand and appreciate, copiously illustrated with examples. It must be kept in mind that the problem of zoning is new to this generation, and needs study. Public and administrative officials should be taught not to be discouraged by adverse Court decisions. Every one who accomplishes anything will make some mistakes. The public and its officials must be taught to meet conditions with fortitude and take steps immediately to correct errors, without discouragement. It may seem that this is asking too much of an engineer. Every man who dedicates his life to a profession should know that his paramount duty is to improve the conditions of the people by education.

To be successful, zoning should meet the requirements of the enlightened public, should be helpful to all classes, applicable and adaptable to the community at the time, and permit adaptation to changed conditions at a future time. Care must be exercised in zoning that orderly, sane, competitive effort in the industrial and business growth is not throttled. Economic forces are inexorable and will not tolerate contravening Utopian ideas. Furthermore, the street system of a city will play an important part in the success or failure of practical zoning. For if zoning over-centralizes industry or business, the streets will become over-congested and transit facilities overtaxed, bringing burdensome and unnecessary expense upon the land, and creating inflated values from which no reasonable return can be obtained.

It is the opinion of many that zoning and city planning are so interlocked and closely allied as to be inseparable. For it is plain that to zone without thought of accessibility and intercommunication would be a fault not easily overcome. On the other hand, to plan without thought of adaptable use of the land would be only the wildest guess. Zoning is not a panacea for all the ailments that a populous community is heir to, but, properly exercised, it will help a great deal.

Zoning a city before the city plan is devised may mean the eventual necessary abandonment of the best solution of the city plan, or it may lead to excessive costs in order to put into execution the best solution for the plan of a city.

Zoning may bring disaster to a city by congregating certain like industries, and thus developing a peak load on inadequate streets, instead of distributing the load by having diversified industries in each section of the city. By the latter means decentralization may be naturally secured. Thus, zoning may best be done after, or as a part of, the general city plan and not before, or independent of the city plan of which it is simply one small part. It might be more dangerous than establishing one of the other elements of city plan-

ning, such as traffic, transit railroad terminals, streets, parks, playgrounds, etc., without taking the others into consideration.

Zoning is invoked to stabilize values; perhaps it will for a time. The stabilization of values means that the price of land will not decrease materially, neither will it increase materially. While this effort is going on, however, what is to prevent the economic forces from becoming active because of some obscure happening, like the relocation of a railroad terminal. Stabilization of real estate values never will be accomplished by zoning any more than will the price of any commodity. It will always be controlled by demand. Zoning is not for the exclusive benefit of the land owner or speculator; it is for the whole people, and for no favored class. Its prime purpose is to help the public obtain the benefit of an efficient, full functioning development, with the least expenditure of time and money.

W. A. Weldin,* M. Am. Soc. C. E.—Pittsburgh engineers have lived long enough under the provisions of the Pittsburgh zoning ordinance to appreciate its excellence and the good work Mr. Knowles has done in drawing up the legislation and in setting up the machinery of enforcement and appeal. The ordinance seems to be popular, or, at least, the general idea of zoning is popular, and it seems that, in spite of the large number of appeals, the legislation is a success.

It occurred to the speaker that there is a small detail in the matter of administration that has apparently been overlooked. In the Pittsburgh zoning ordinance, cemeteries are prohibited in residential sections by omission from the list of permitted uses of land. Obviously, the enforcement of this prohibition cannot be lodged with the building inspector. Some other agency will have to be called on. Perhaps the solution is with the Department of Public Health, which must give a burial permit. Perhaps this permit could be made to specify the place of burial, and so give some officer a chance to prevent illegal extensions of cemeteries.

It seems to the speaker that the large number of appeals in Pittsburgh is accounted for by the rugged topography, which creates so many difficulties for the builder. On this account, it is necessary to give the Board of Appeals a wide range of discretion. This is a point that should not be forgotten in drawing up zoning ordinances.

Planning commissions and similar bodies have been much criticized for delay in rendering decisions. This is perhaps the greatest source of antagonism, especially on the part of those who are engaged in subdividing land, and in building. They feel they are the ones who are actually building the city (probably this is true as to the actual determination of what is to be built), and that they should be encouraged and helped, and not be subject to restrictions and delays. Delays in many cases will absorb all or a large part of the profits of an undertaking. This criticism does not apply to the Board of Appeals of Pittsburgh, which is acting promptly on whatever comes before it.

The information on the fire hazard in tall buildings given by Mr. Crane is interesting and would have been still more so had it presented data as to

* Civ. and Min. Engr. (Blum, Weldin & Co.), Pittsburgh, Pa.

the actual incidence of fire in such structures. Unless it can be shown that fewer fires, or smaller loss of life and property occur in 10-story buildings of modern type than in tall buildings, the alleged difficulties of fire-fighting would seem to be without application to the point under discussion.

In regard to the physiological effect of tall buildings on the inhabitants of a city, it seems to the speaker that Mr. Crane's assumption that dark and poorly ventilated rooms are adjuncts of the skyscraper is unwarranted. On the contrary, the tall buildings give a maximum of light and air for the workers therein. The great advantage to the health of the students offered by the proposed 52-story building of the University of Pittsburgh, as compared with the alternate scheme of 14 acres of 4-story buildings, is one of the strong points urged in favor of the tall building project. That a reduction of light and air on the street surface makes the streets "disease-breeding canyons" seems a rather exaggerated statement.

As to economies of the tall building there are too many of these structures built by banks and estates from trust funds, to substantiate the fear that such investments are deliberately and repeatedly made only to produce an annual net loss. The University of Pittsburgh project it is said on the authority of well-known engineers will result in a sufficiently lower operating cost to more than offset the slightly greater construction cost of the high buildings.

If the speaker is correctly informed, buildings of twenty stories and more are being financed in several cities by bonds, which are redeemed, principal and interest, serially out of rentals. The mere fact that such bonds are thus paid off would seem to dispose of the "losing venture" theory. Besides, if the very high building is an economic fallacy, would it not be simpler to publish the figures, to stop further construction, than to legislate against it?

It seems to the speaker, that the ideal to be approached is tall buildings properly spaced, rather than dense masses of lower heights. The indicated legislation, then, would aim to prevent the very uniformity that a mere height limitation would create.

Cannot an ordinance be drawn, extending the principle of the New York law so as to require a set-back not only on the street front, but along all property lines; in other words, to permit a tall building only in case the prospective builder owns a lot of specified greater area than the base of the proposed tall portion?

Rudolph P. Miller,* M. Am. Soc. C. E.—Most of those who discuss zoning treat it as something new. Mr. Edward M. Bassett, who is the leading authority on zoning, in a recent paper on its constitutionality, speaks of it as a new science. It should be considered not as new science, but rather as a new application of an old principle. The principle has been applied for many years in connection with the establishment of so-called fire limits as far as the construction of buildings is concerned, that is, districts in which the erection of frame buildings is prohibited. In this respect the principle has been sustained by the Courts in many decisions. It seems surprising, therefore, that the Courts at the present time are regarding zoning as a new principle. There

* Cons. Engr., New York, N. Y.

have been some cases in which the Courts have held that a city could not fix fire limits unless it had been given specific authority to do so by statute, but since that time the principle seems to have been well recognized as an established practice, even without special authorization by statute.

Adverse Court decisions with respect to zoning have been largely because, under the guise of zoning, attempts have been made to do things that cannot be considered as in the interest of the public health, safety, or welfare.

Furthermore, too much differentiation is attempted in some zoning ordinances, even between one-family and two-family houses. There was a case recently in New York State in which an attempted differentiation was set aside by the Court. A sorority building was to be built in a residence district. The enforcing officer felt that a sorority could not be considered as a family. The Court held that there was no more harm in the erection of a sorority building in a residence district than of an ordinary dwelling. There would even seem to be some doubt as to whether a differentiation between one-family and two-family houses and multi-family houses will stand the test, although sentiment generally is so strong in favor of such distinction, that the Courts would probably take the public view of it.

Unless it is established that a given use in a district is clearly a nuisance, the Courts will not sustain its prohibition. In some cases, it has even been necessary to wait till the use has been established and proved to be a nuisance before the Courts would restrain its continuance in any particular district.

On every board of appeals, or board of adjustment, there should be an engineer. The personnel of such a board, however, should not include any official or any person who is connected with the formulation of the rules or laws, nor one who is engaged in the enforcement of those rules. An appellant before the board, if he must face the same officer who has refused him the permit, feels that his case is prejudiced at the start. There is no reason, however, why the enforcing officer should not appear before the board to state fully his position in the matter.

As to the limitation of height, it has been suggested that provision ought to be made to allow a building here and there to be erected to a considerable height above the others. The New York law fairly well provides for this in the section which allows the erection of towers to unlimited heights, provided that in plan they are not more than 25% of the area of the lot. This makes it possible for two or more owners to combine, as they sometimes do, in the erection of buildings, and to erect a tower of considerable area and unlimited height, or any height that might be considered economically sound.

Leonard S. Smith,* M. Am. Soc. C. E.—The remarkable growth of city planning and zoning in American cities is undoubtedly the outstanding civic development of the present day. The realization of these plans, however, has been greatly handicapped through a lack of understanding by the general public with a resulting fatal lukewarmness toward carrying them out. Many a city has a plan, but lacks the necessary leadership to act on it.

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The great universities are taking an important part in supplying this needed understanding and future leadership; for example, at the Engineering College of the University of Wisconsin, courses in City Planning have been given for about fifteen years. During the last six years, 1 250 students have taken one or more of the three courses offered. More than half these students have made investigations of the city planning needs of their home towns and have incorporated them in well prepared and illustrated themes. In addition to engineers, these students included those having economics, sociology, journalism, and landscape architecture as their major subjects.

For the most part they become much interested in city problems, and judging from letters received by the speaker, continued their interest after graduation. It would not seem to be a vain hope to visualize many of these men and women as future leaders of civic improvement in their home cities; in fact, examples of this have already been noted.

Probably many other universities are making similar records and, if so, the aggregate influence of these graduates will eventually be an important factor in the problem of getting things done at the right time and place.

The obvious need for engineers to take a greater interest in civic movements and to furnish leadership has been emphasized by numerous writers. In no better way can the profession secure a well earned appreciation than by giving this type of public service.

Albert P. Allen,* Esq.—The telephone business is one of perhaps a small group whose daily bread may be said to depend on the unanswered question of zoning and its actual solution, either with or without proper ordinances. In other words, for many years, this industry has been trying to anticipate what the actual zoning in Chicago was going to be; this also is true in connection with all the principal cities in Illinois. For reasons indicated in the speaker's paper,† entitled "the Influence of Zoning on the Design of a Telephone Plant", a telephone company has to look further ahead in connection with the distribution and character of population than the other utilities, and it is a fact that the work it is now approving and starting is dependent to a great degree on estimates of what the conditions will be in 15 or even 20 years. As a matter of fact, by the time these 20-year estimates are 5 years old, the plant engineers will be asking for a new 20-year estimate, because some of the problems which arise in connection with plant extensions, office locations, etc., cannot be satisfactorily answered without such information.

Moreover, this is a practical proposition with the telephone company that affects, to the extent possibly of millions of dollars, the ultimate plans for its growth and extension. In Chicago, for instance, to maintain adequate service requires, and, apparently, will continue to require, $10 000 000 or $15 000 000 per year. The same, in proportion, is true of Detroit, Mich., New York, N. Y., and of all large cities; for this reason the telephone companies are directly and intensely interested in knowing the future distribution and character of population. It is vital to them to know where to locate operating centers, how

* Commercial Engr., Illinois Bell Telephone Co., Chicago, Ill.

† See p. 692.

big to make the conduit runs, and what size of cables to plan to use. This makes it important to know, for instance, whether the height of buildings in a certain area is to be limited to 10 stories, to 20 stories, or higher.

In connection with this much discussed question of the proper height of buildings, some of the conflicting opinions and the reasons therefor are amusing. For instance, facts are presented to show that very tall buildings are uneconomical, and then it is argued that those who are allowed to build them are thus given a commercial and economical advantage. It seems probable that if too many high buildings are erected in any area the public would object and decide to locate its offices in other parts of the city where high buildings do not exist. As a matter of fact, however, even in New York and Chicago a limited portion of the total area has been built up with very high buildings. An aeroplane photograph of Lower Manhattan or of the Chicago Loop District will illustrate this truth. Less than 3% of the area of the Chicago Loop is built up to 16 stories or more and yet the privilege of building as high as that has existed for many years. The average height in the Chicago Loop is less than 8 stories and regardless of legal permission a great deal of that area, as in Lower Manhattan, will never be built up to more than 5 or 10 stories.

However, if the height of buildings is to be legally restricted by zoning ordinance, or otherwise, the telephone company wishes to know the facts as soon as possible. It is much interested in the probable outcome of these zoning regulations in their effect either on the distribution of population, the height of office buildings, apartment buildings, or on the number of families that eventually will be allowed or can be expected in various sections of the city.

E. P. Goodrich,* M. Am. Soc. C. E.—The thanks of the membership for real contributions on the subject of City Planning are due those who prepared these papers, each of which stands out in a different category. Any engineer who goes before a Court in a particular case can probably turn to these papers and say, "This man, an expert on that particular subject, has said thus and so". For example, the late Professor Whipple discussed a subject which is of the utmost importance, that is, health with relationship to zoning, and his paper unquestionably will be of value in its several aspects. There are, however, one or two items in the paper that are somewhat in error, for instance, in regard to three-quarters of the ground area of the city being devoted to pivate use.

Professor Whipple also mentioned the desirability of establishing a relationship between size of buildings and area of streets on a scientific basis, which was also discussed by Mr. Crane. That is something in which engineers particularly should have a part. In the past four or five years, the authorities of certain cities have undertaken to make this determination. In connection with the Plan of New York, very extensive investigations as to this relationship are being made.

It would be of great value if each one who can do so would make detailed traffic studies to determine the average length of travel of vehicles, the distribution of variation in length of trip, the average length of the walk of the man going from his place of sleeping, for example, to his office, or from his subway

* Cons. Engr., New York, N. Y.

station to his office. These factors determine the density of street use and the widths of sidewalks and pavements; they re-act to determine in a definite way the proper height which should be placed on buildings. Fire prevention should also be an important factor in any logical study of building height limitation, but the economic factor is really the determining one, as it is in reference to all other phases of zoning. The psychological basis for the sentiment in favor of zoning, if it be analyzed, is doubtless an economic one. The man who owns a single-family house wants it protected. He does not care subconsciously so much about the health of his children as he does about the value of his property. He gets out into the open for the benefit of his children and, thereafter, he wants the value of his property considered. The Courts, however, will not take into account the economic foundation that is the real basis of the desire. They will generally sustain reasonable restrictions, on health and on moral grounds, but, in the background, there is always this economic problem.

Mr. Crane refers to the height of buildings in Chicago, Ill., and the studies made by Mr. George C. Nimmons. A corner lot valued at $1 000 000 was used in this investigation, with building heights varying from 5 to 30 stories. He found the net return on the investment to be as follows:

Stories.	Percentage.	Stories.	Percentage.
5	4.36	20	7
10	6	25	6.72
15	6.82	30	5.65

There is, therefore, a clear-cut maximum rate of return on the investment, on the assumption that the land value is placed at $1 000 000, and that building heights vary. Normally, the land value for a building of that kind is equal to the value of the building itself. Unless that is the case, the depreciation in the building will increase faster than the enhancement in the value of the land, so that there is a gross loss to the owner. Now, if Mr. Nimmons' figures are revised to take into account land values which are equivalent to building values, the results are as follows:

Stories.	Percentage of Profit.	Stories.	Percentage of Loss.
5	3.6	20	0.2
10	1.2	25	1.1
15	0.04	30	1.7

In other words, wherever the proper economic relationship exists, high buildings are not a good investment, and it is always a surprise (as Mr. Bartholomew has pointed out) that insurance companies and bankers will loan on such types of buildings. It is well known that normally they are erected either as an advertisement or as a monument.

The point brought out by Mr. Crane regarding 20-story buildings and 4-story buildings adjacent to the Loop District in Chicago, Ill., needs a little amplification. Mr. Crane states that where there are a certain number of 20-story buildings, with a limitation of the street use as to the pedestrians, in that same vicinity there will be a corresponding number of 4-story buildings.

In consequence, the average of about 8 or 10 stories will be maintained for reasons of broad economics, because the community as a whole barely gets a return on its investment, although a few individuals net an extra return. Land values throughout the Loop District are practically uniform, and it is a question as to what building height will just support the value without starting a vicious circle of inflation.

Mr. Bartholomew brings out an interesting fact relative to zoning, namely, that area regulations have not been much discussed. The reason is that those who draw the regulations have not made them stringent enough. The case of an Omaha church is, possibly, the only one in which an area regulation has been brought into Court. In that case, the original regulation was upset because the church was in a "residence" district, and the restrictions were too rigid in the Court's view for that particular structure.

The important relationship between land subdivision and zoning was also stressed by Mr. Bartholomew. In several places in the Middle West, city planning commissions have secured the simultaneous promulgation of the zoning ordinance and of rules and regulations for platting. Furthermore, they have either formulated new building codes at the same time with the zoning ordinance, or provided, at least, that the zoning ordinance should be fully co-ordinated with the building code and, similarly, with the housing code.

The statement by Messrs. Sheridan and Hoffman that streets cannot be legally widened under building line zoning should be emphasized. Nevertheless, it is an excellent means of securing indirectly what is desired. Sometimes it can be accomplished, but only rarely. Building line establishment, therefore, has been effected almost always by eminent domain.

As to Mr. Whitten's paper on "Housing Density Regulation" it is felt that the stipulation of a number of families per acre, or a number of square feet per family, is unnecessary, is simply an encumbrance on the ordinance, and is difficult of administration.

Provided there are yards of proper size (of the types outlined by Mr. Bartholomew), plus a building height limitation and a proper use differentiation, sufficient data are at hand to limit automatically the families per acre, and thereby to distribute the congestion. With adequate sunlight planning, with proper sizes of yards and courts and widths of streets, and the control of room densities of population, a fundamentally sound basis is established for scientific zoning and building construction.

The paper presented by Mr. Knowles is so good that it is difficult to discuss it. Most Boards of Appeals have no engineer member. Generally, either an engineer or an architect is stipulated, and because the architects are usually more numerous (if not more powerful) than the engineers, the architects get the appointment.

It seems to the speaker that both Pittsburgh, Pa., and Rochester, N. Y., have had a large number of appeals as compared with other cities, and a correspondingly large number of appeals granted, and if it were not for the personality of the individual members of the Boards the situation would be open to grave question.

This brings out the fact finally that the whole question is one that may be called "human engineering." Perhaps it is not yet possible to say that yards and courts should be of explicit sizes, due to sunlight and the reacting effect of sunlight on health (tuberculosis, rickets, and other diseases) as outlined by the late Professor Whipple, but the speaker believes that by analyzing the psychology of city planning it will be clear that zoning is an engineering problem for human engineers.

MORRIS KNOWLES,* M. AM. SOC. C. E. (by letter).—As has been well said, this group of papers and the discussions thereof may serve as a useful compendium of the fast developing subject of zoning.

Of these treatises, that by the late George C. Whipple, M. Am. Soc. C. E., entitled "Zoning and Health", and his previous paper, "Sanitation—Its Relation to Health and Life",† delivered before the Sanitary Engineering Division on January 15, 1924, comprise within themselves an excellent brief on the whole subject of zoning. In the same connection there is a similar paper by Professor C.-E. A. Winslow entitled "The Atmosphere and Its Relations to Human Health and Comfort",‡ delivered as an address before the Sanitary Engineering Division on January 20, 1925. These papers comprehend arguments for, and statements of, good results which will come from zoning wisely undertaken and in accordance with good principles. Any one interested in the subject and honestly desirous of learning what is reasonable and what may be fully warranted under the law, with regard to its effect on public health, will be greatly repaid by reading these excellent presentations. The appellation "collective self-control" is an excellent coinage for a subject that means so much to present-day collective and co-operative life.

The paper by Mr. Fisher is of interest because of its description of the unique method used in Rochester, N. Y., a city which has taken such an advanced position on so many civic subjects. The other papers, each by a man prominent in the profession and familiar with the development and application of zoning, are important contributions and worthy of close study. Perhaps it has been seldom that, of a subject which is so new in the art, there has been so complete a group of papers so early in the history of its development. It bespeaks much for the public interest, for the activity of the engineer, and for his willingness to compare notes and to help in the formulation of principles.

The writer is much gratified at the friendly and co-operative discussion of his own paper, with regard to the purpose and methods of administration of zoning. The prevailing tone, voicing the necessity and importance of co-operation and the high-minded civic spirit which must be inculcated and observed, is a growing indication of the engineer's breadth of view and of the fact that his counsel is regarded and observed on matters not strictly technical within the engineering field.

As Mr. Crosby has well stated, because of technical knowledge and a realization of the effect of the application of zoning, both good and bad, the respon-

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

† See p. 87.

‡ *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 794.

sibility for educational work does rest heavily on the Engineering Profession. Mr. Tuttle emphasizes the need of conservative methods and consideration of the subject along broad lines and within common-sense limits.

Mr. Leavitt and Mr. Weir express this same thought in another way when they speak of the need of recognizing the helpfulness of education. Nothing is more important than a thorough-going appreciation that zoning is only a part of the broader subject of city planning, and these two discussors do well to call attention to the harm which may come from attempts to zone first and plan afterward. Greater expense and disaster to the well-being of the community may follow the attempts of the opportunist who would first fix future growth before he has wisely determined its tendencies.

Mr. Weldin has touched on an important matter, namely, the omission of cemeteries from the list of permissible uses in the residential sections; but he wisely calls attention to the fact that these appear to demand special treatment. The areas required are usually quite large, selected a long time ahead of immediate complete use, and, therefore, are more directly a part of the layout of the city plan, and when so laid out, if subsequent to the enactment of a zoning ordinance, they will call for the necessary amendment thereto. Their particular character seems to call for special treatment. Mr. Weldin is over-praiseworthy with regard to the authorship and responsibility of the Pittsburgh ordinance, for if there ever was a document which has been the result of the application of many minds the Pittsburgh ordinance is that instrument. Co-operative effort, harmony, and unity of support seems to have resulted in the production of an excellent working document.

Mr. Miller emphasizes the well-known fact that, all too frequently, adverse decisions come from unwise attempts to do the ill-considered thing. The writer cannot agree that every Board of Appeals should have an engineer. It is wise, however, that there should be a man versed in the understanding of building and construction, who can bring to the Board the advantages of his point of view and his aid in reaching solutions that will promote accomplishment and, at the same time, not injure his neighbors. If such a man is an engineer, it is gratifying, but above all he should have had building experience and this may be obtained in professions other than engineering.

The example of a course in City Planning at the University of Wisconsin, described by Professor Smith, is typical of the growing feeling that zoning, and the more inclusive subject of city planning, is a proper study, not only for students in engineering, but for those in liberal arts. It is not all-important that the student shall subsequently practice this particular field, but his views will have been made wider and his thoughts stimulated along the lines of civic development. He will be a better citizen and will be able to act as a more capable executive and administrator.

Mr. Goodrich well calls attention to the fallacy of the belief that the higher the business building the more it will pay as an investment. Time alone can correct misapprehension on this point, and it is unfortunately true that many an investor, bank, or municipality, must thus painstakingly realize

the futility of expecting to pile story on story in the business districts and still be able to transact affairs within the limited congested area.

The gratifying response and thorough presentation in all these papers, and the exhibition of the ramified relationship between zoning, city planning, and all forms of public activities, is a marked step in advance of that which would have occurred among a similar group of technical people a generation ago. Civilization indeed moves forward, and it is pleasing to note that the engineer is doing his share toward such advance.

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Paper No. 1564

THE INFLUENCE OF ZONING ON THE DESIGN OF PUBLIC UTILITIES*

A SYMPOSIUM

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* Presented at the meeting of the City Planning Division, Detroit, Mich., October 25, 1925.

THE RELATION OF ZONING TO THE DESIGN OF DRAINAGE AND SEWERAGE SYSTEMS

By Paul Hansen,* M. Am. Soc. C. E.

The relation of zoning to drainage and sewerage is so obvious and direct that little need be said on the subject to convince one that an established zoning system facilitates design and favors economy.

Zoning, which may be assumed to be a product of city planning, if begun in time, may prevent low-lying land, subject to floods or troubled with high ground-water, from being occupied in a way that requires drainage, thus obviating sewers on excessively flat grades and, in many cases, also pumping. Flat grades and pumping result in high costs not only for construction, but also for maintenance and operation. If low-lying areas are permitted to be utilized for dwellings they become as a rule low-grade property areas and, therefore, are unable to bear the heavy assessments required for a proper sewerage and drainage system. As a matter of fact, they rarely are provided with a proper installation, and if they were a large part of the cost is likely to be saddled on the community as a whole.

Although utilization of so-called waste and unusable areas should not be the criterion for the location of parks, low-lying areas generally lend themselves well to the development of parks with lagoons, brooks, and other interesting features, and when thus developed afford particularly attractive scenic effects for residence property on the higher ground bordering them.

It is not always practical or desirable to acquire or utilize low-lying areas for parks, but it is generally possible to restrict or zone these areas so as not to multiply the difficulties attendant on poor natural drainage. For example, such areas may be adaptable for use by railroads and heavy industries that can afford to raise the natural levels by means of fills. The wastes from some industries such as steel works and power plants make good filling material. Such utilization does not require extensive sewers and underground drainage, and if it should the value of the properties is usually ample to stand assessments sufficiently large to provide adequate relief.

Although not always the case, a well conceived zoning plan may prevent, prematurely at least, the extension of built-up areas into new areas that cannot readily be drained into the common outlet. Extension of communities into adjoining drainage areas may require separate sewage treatment works, always an undesirable arrangement, especially if such treatment works are small and remote; or it may require costly pumping to deliver the sewage into the main drainage area.

The tendency to spread unreasonably over large areas, leaving numerous vacant lots which are unsightly and an economic waste, is a striking feature of a community without a zoning ordinance. This tendency is more likely to prevail in relatively level areas where the laying out of streets is a com-

* (Pearse, Greeley & Hansen), Chicago, Ill.

paratively simple problem and little or no grading is required. Chicago, Ill., presents a conspicuous example of this tendency.

The vacant-lot evil is brought about usually by the encroachment of stores, apartments, and industries in localities originally developed as single-family residence districts. The result is a decline in land values and a decrease in building operations which may extend over many years.

Meanwhile, sewerage and drainage will have been provided, involving relatively high assessments on property which has failed to achieve the increase in value that might normally be expected, or which may actually have decreased in value.

With zoning, property values become assured, and there is a virtual certainty of continuous and steady development with the temptation for land speculation largely removed until the entire area is fully utilized in the manner, and for the purpose contemplated. Thus, it is highly improbable that there will result the economic waste of idle land loaded down with the carrying charges for sewerage as well as other improvements.

The most direct effect that the engineer realizes in undertaking to design a sewerage system in a community which has been zoned and which has an effectively administered zoning ordinance, is the relative certainty and economy with which he can adapt the sewers to the service they will be called on to perform. It is also well known that combined sewers and storm-water sewers can rarely if ever be provided with sufficient capacity to remove immediately maximum rainfalls. The perfection of sewer performance must be proportioned to property values. In zoned areas the property values are relatively easy to determine and also relatively stable, so that the engineer has a much more reliable guide to judgment than he has where the future character and, therefore, the future value of the property is uncertain. It is further to be noted that with more extended use of the zoning system, more and better data will become available.

An illustration of the advantage of zoning occurred in the speaker's practice at Decatur, Ill. In 1920, Mr. Myron H. West prepared a city plan with which a zoning system was proposed. It then became apparent that an extensive revision of the sewerage system was necessary, and the speaker's firm was called on to design a main drainage system to care for present and future requirements, utilizing the existing sewers as far as practicable. With certain areas designated and assured for certain purposes, it was comparatively easy to determine what allowance to make for such items as percentage of impervious areas, depth of sewers required, and volume of sanitary sewage and liquid industrial wastes. By knowing the general value and character of the property to be drained, it was also possible to form a reliable judgment of the magnitude of the storms to be cared for; in other words, how much it was wise and proper to expend for sewers.

Figs. 1 and 2 show typical blocks in zones of different characters. There is also noted on the diagrams the percentages of pervious and impervious areas and the population per acre. These diagrams present what are believed to be average conditions. They are by no means final, and criticism of them in the light of further experience with zoning practice would be interesting.

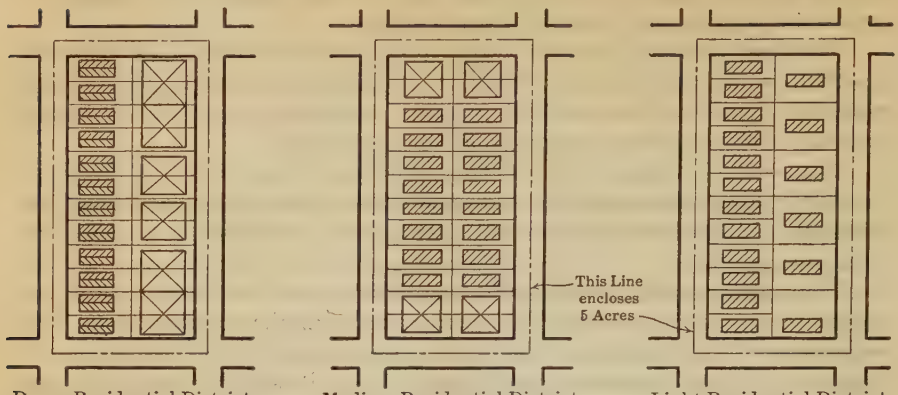
This Line
encloses
5 Acres
Dense Residential District
Total Population 275
Density - 55 per Acre
34% Impervious
Medium Residential District
Total Population 175
Density - 35 per Acre
27% Impervious
Light Residential District
Total Population 75
Density - 15 per Acre
20% Impervious
One Family Dwelling — 5 Persons
Two Family Dwelling — 10 Persons
Six Family Apartment — 24 Persons
TYPICAL BLOCKS
FOR
DIFFERENT KINDS OF ZONES
SHOWING
PERCENTAGES OF IMPERVIOUS
SURFACES AND ASSUMED
POPULATION PER ACRE
CONTRIBUTING DOMESTIC SEWAGE

FIG. 1.

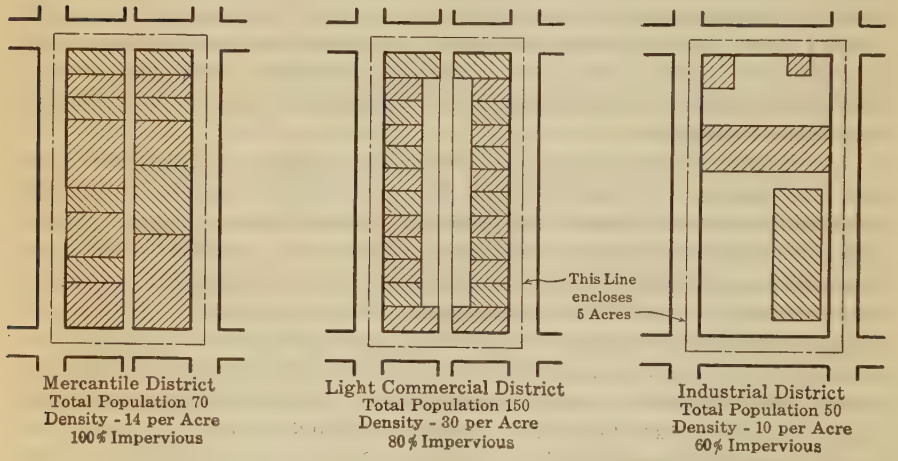
This Line
encloses
5 Acres
Mercantile District
Total Population 70
Density - 14 per Acre
100% Impervious
Light Commercial District
Total Population 150
Density - 30 per Acre
80% Impervious
Industrial District
Total Population 50
Density - 10 per Acre
60% Impervious
Lots, 50 ft. wide
Block, 330 by 660 ft.
TYPICAL BLOCKS
FOR
DIFFERENT KINDS OF ZONES
SHOWING
PERCENTAGES OF IMPERVIOUS
SURFACES AND ASSUMED
POPULATION PER ACRE
CONTRIBUTING DOMESTIC SEWAGE

FIG. 2.

The question of the severity of storms to be provided for in designing the sewers within the various zones, a question which reduces to one of how much may advantageously be spent on sewers as insurance against the inconvenience of flooded streets and basements, is difficult to answer. There is no direct or precise way of finding an answer, because it is so difficult to equate the cost against the probable inconvenience and damage. Experience must be relied upon to crystallize practice. This, in fact, is going on at the present time, and practice in the larger cities, while not greatly divergent, is more variable than it should be and is more or less lacking in logical procedure.

Usually an assumption is made that the sewers must be capable of removing promptly a rainfall not likely to occur more often than once in so many years. This method of approach will continue, no doubt, to be the basic method for determining the size of sewers required, and increased experience with zoned areas will greatly improve engineering judgment along this line.

The taxpayer and the municipal official, however, are strongly inclined to consider price, and, therefore, it becomes important that a maximum price per unit of area or per front foot of property must be placed on securing an accepted standard of service. Conditions vary in different localities; a figure per front foot that would easily purchase adequate service in one place, because of good grades and pervious soil, might be inadequate in a locality requiring flat grades and having an impervious soil. Nevertheless, in the larger populated centers, there is not so great a "spread" in conditions, but that figures for the mean price per front foot for complete sewerage may be developed for different zones, which will constitute a fair measure of good and adequate practice. Allowances for departures from the established maximum can be made from these mean figures on the basis of local conditions. Here, again, the wider adoption of zoning will help build up a sound practice that will enable engineers to solve the economic phases of sewer design with greater precision.

In a paper before the Western Society of Engineers, S. A. Greeley,* M. Am. Soc. C. E., has suggested that the front-foot cost of sewer service be a measure for establishing what might be called a sewerage policy, and he has assembled costs from various cities mostly applicable to residence areas, because at present these are the only figures available. In general, it was found that, at present costs, the trend is toward $5 per front foot for complete sewer service. In the future, it will be desirable to study and correlate figures relating to different classes of zones.

From any aspect, the engineer who is concerned with sewerage and drainage can only welcome the wider adoption of zoning ordinances. They mean better sewerage for less money.

* "Notes on the Relation Between the Capacity of Combined and Storm Sewers, Their Cost and the Frontage Assessment", *Journal*, Western Soc. of Engrs., Vol. 30, January, 1925.

THE INFLUENCE OF ZONING ON THE DESIGN OF THE STREET SYSTEM

By T. Glenn Phillips,* Esq.

Zoning is part of a rather recent and refreshing phase of city planning practice, a phase in which city planners are relaxing rather hopeless efforts to remodel old cities to fit modern needs, and are turning to the more fruitful endeavor of making present-day requirements fit the cities. When the remodeling process has come up against a blank wall—and that is not a mere figure of speech—for further improvement, more intelligent and efficient use of the existing facilities has necessarily been sought. This is true particularly of streets and street traffic. After the utmost has been done in the removal of encroachments and the widening of pavements and there is no way of providing more roadway space except by taking property too costly to take, methods have been devised for handling increased traffic on the existing roadways, often with astonishing and creditable results.

A further step may be taken in continuation of this same process, one which a few far-sighted city planners have dared to suggest, but which to the best of the writer's knowledge no American community has dared to undertake. That step is the control through zoning of the causes of street traffic, the places where people want to go and from which they come, so that no one property or locality will be the point of origin or destination of more traffic than the streets serving it can economically bear.

Zoning has become generally accepted as a logical and reasonable method of controlling the use of property in the interest of the community, but is generally regarded as a device for protecting one property owner against damage from any inimical use to which neighboring property might be put. That zoning may be used with equal reasonableness to protect the streets against action of individual property owners, which may render them less useful to the community, is not so generally realized. Yet, it is only an amplification of the first idea, because if one man uses his property so that he causes traffic congestion which renders the street less useful to his neighbors, or requires an expense for measures of traffic relief not otherwise necessary, he is damaging that neighbor and the community just as surely as if he "hogged" all the light and air or erected a glue factory.

The effective use of city property depends on the use of the streets. If a city lot has no means of approach, it is useless. Its value is measured by its accessibility, and if the effective use of the streets is impaired the value of the property depending on them for access is also injured. When this impairment is due to an excess of traffic over maximum street capacity, there are only two remedies: One is to increase street capacity by multiplying the surface area vertically within street property or horizontally over property previously

* Consultant, City Plan Comm., Detroit, Mich.

devoted to some other use; the other remedy is to restrict the use of property abutting on the streets to the point where the traffic productiveness of each parcel coincides with its share in the traffic capacity of the street. The former is the method generally used where a remedy is attempted; but used alone it is not a remedy—it is simply a postponement. The latter method, however, does provide a real remedy if taken in time, and, if not, may be coupled with the first to make it effective.

It may be well to state before going further that the writer is not using the term "street" in a narrow sense, but broadly to cover all channels by which people or goods proceed from one point to another within a city. This is justified by the fact that the ordinary forms of sub-surface or overhead transit are merely supplements to the street system, and if (as is rarely the case) provided with rights of way independent of street property, are looked on as potential streets whenever traffic demands may cause their taking and proper treatment to provide additional roadway space.

The restriction of the use of property in the interest of advantageous use of the streets may be accomplished under zoning in two ways, first, by limiting the intensity of property use to fit street capacity; and, second, by distributing the kinds of property use to fit the street layout. The first is bulk zoning, the second, use zoning, and, for convenience in discussion, they will be treated separately. Both are applicable over the entire city, but the former has its particular field in the built-up sections where traffic difficulties generally appear first and where the character of the property use is already determined beyond the power of zoning to change materially.

In the down-town sections of large cities, the balance between street property and private property is generally struck well before traffic problems become acute. Property values and types of buildings abutting on principal streets are such that substantial widenings are seldom practicable, and although by-pass routes through back property, along old canal beds, or over railroad rights of way, are often securable, their improvement is costly.

The method of relieving traffic by substituting subways for surface transit has already proved a boomerang, and will continue to do so wherever it is used without rigid zoning restrictions on the height and bulk of buildings in the area affected by the improvement. Every facility for increasing the traffic capacity of a street increases the accessibility of the abutting properties and directly enhances their value. The owners of those properties are seldom slow in taking advantage of the new situation by erecting more commodious buildings which, in their turn, represent enlarged traffic quotas, soon overtaxing the new facilities and demanding still further improvement. As the process goes on, the cost rises disproportionately because the simpler remedies are applied first and the more drastic are deferred, while the costs of necessary property takings and complications in construction are constantly increasing the expense of their realization.

Without the interposition of zoning, the city is powerless to stop the recurrence of these cycles, for its control extends only to the facilities for traffic and not to its causes or results, and even that control is impotent because it is responsive to popular clamor which takes account of little beyond the demands

of the moment. That the cycles bring only increased congestion, less daylight, and less wholesome air in city life, and an ever heavier financial burden on the public, is sufficient reason for bringing them to a stop.

Zoning embodies the power to do this, but gradually and without the use of the proverbial "monkey wrench". Intelligently applied, it gives notice far enough ahead for the community to adjust itself to the ultimate limit before it is reached. Zoning is based on conditions as they are, it has no power to destroy and build new, but brings improvement by preserving the good and preventing recurrence of the bad. It does not stop growth, but merely directs it along healthy channels.

When the framing of a zoning ordinance is undertaken, the bulk requirements for the down-town sections must be given particularly careful study because of the high property values represented, on the one hand, and because of the vital community interest in adequate approach to and circulation within the district, on the other.

First must come a critical examination of the existing street system in order to determine if it is the best obtainable. There is no advantage in shielding features obviously bad or in preserving deficiencies. Zoning does not correct; its action is preventive. If healthy future growth is assured through its action, it is only logical that any rottenness at the core should be removed before the new growth is added.

There will be situations in which the life of the community has so grown about the defective parts that correction is a major operation. This brings on the next step, an examination of each such case to determine whether the improvement can be made within a cost and with such benefits to the community as to make it economically justifiable. There is often room for nice determination whether the annual charges incurred by the execution of an improvement will not exceed the annual loss arising from its non-execution measuring cost, of course, with due respect to amenities as well as finances. Zoning in itself cannot be relied upon to take care of a bad traffic situation, but in a case where traffic is approaching, but has not exceeded, the capacity of its arteries, the action of zoning may check the process of congestion ahead of the point at which radical measures of relief will be demanded. If the situation has passed the point where, say, the construction of a subway could be avoided, then zoning can be applied on the basis of the subway's existence, but with limitations designed to prevent duplication of the subway facilities at a later date. If a radical change in the traffic capacity of the street system is necessary, either through the opening of new streets or the building of subsurface transit lines, no zoning limits should be set which will prevent the full economic use of the new facilities, or if property owners are locally assessed for the improvement, deny them the opportunity to recoup the cost by more intensive use of their property.

These studies comprise nothing more than a determination of the best and most economic street system obtainable on the assumption of zoning powers adequate to preserve the relationships between traffic production and traffic facilities on which it is based. Ideally, zoning should come first and the street system be designed to serve the various types of district provided for, but such

action is limited to the undeveloped fringes of cities in practice, and then only to territory within the municipal boundary. With the main framework of the street system established, the province of zoning is to make the best of it, and to prevent such abuse of the facilities by overtaxing their capacity that a re-organization economically disadvantageous to the community will be required.

Once this existing and potential street system is in mind, the framing of the zoning plan to fit it can be undertaken. The relationship between the size of various types of buildings and the traffic for which they are responsible is capable of fairly accurate determination. The number of occupants or daily visitors per unit of floor area can be found by averaging conditions in existing buildings of the same type. The movement of materials to and from the buildings is similarly determinable. The proportion of persons coming by automobile to the door, by transit lines to the immediate vicinity, or on foot from transit line stops or stations at a distance, can also be calculated with fair accuracy, factors being the size of the city, the ease and speed of approach by the different methods, the percentage of car ownership, and future tendencies. A very interesting and thorough study of this kind was made for Cincinnati, Ohio, in 1923, by the Technical Advisory Corporation under the direction of E. P. Goodrich, M. Am. Soc. C. E. On the basis of the usual sidewalk width in the down-town section, the average sidewalk capacity in persons per minute, and the quota of the sidewalk crowd which each office building would have to absorb through its elevator equipment, a figure was computed for the maximum height of buildings permissible if the customary occupants had to walk one, two, or three blocks from the transit lines. The figures obtained, of ten stories on the basis of an average walk of one block, and of only five stories, if that walk were lengthened to three blocks, are sufficient to indicate how far the usual zoning ordinance fails to restrict down-town buildings to street capacity. These figures were based on a 90% utilization of ground area and uniform building to the maximum height. The latter condition is rarely found where tall buildings are permitted, the average for the typical skyscraper district being surprisingly low, but under more drastic restrictions it is probable the maximum would be reached by all new buildings, and that right at least must be accorded all property in the zone under an ordinance presuming to be fair and democratic.

It will be noted that in the study just cited, sidewalk capacity was used as the controlling factor. In down-town office and shopping districts, the bulk of the users approach the buildings on foot, often overflowing the entire pavement area in rush hours. The number of private vehicles using the down-town streets in peak hours is very responsive to traffic conditions, for when the use of the streets is inconvenient, those who use their cars solely for going back and forth to work abandon them in favor of the transit facilities. This leaves the vehicular traffic on the down-town streets composed principally of four elements, trolleys and busses, through travel, necessary commercial traffic, and the cars of shoppers and theatre-goers. The last can be largely discounted, because it comes between the peak hours of the first, except for late shoppers and matinee patrons.

Through travel as found in any one locality, need not be travel passing entirely through the city, but may be traversing one section en route to another. There will generally be proportionally less of this in the central business section than elsewhere, because a larger share of the traffic entering it finds its destination there. The proportion between through traffic and strictly local travel in any district can be determined by traffic counts at the natural gateways to the district, license numbers and time of passage of cars in and out being taken and checked in order to determine how many stay in the area only long enough to effect the passage through it. The thoroughfare system of each city and the location of the various business and industrial areas, freight yards, wharves, etc., will determine how necessary it is for traffic to pass through congested areas en route to points beyond.

If by-passing is possible, it will be used in proportion to the difficulties of direct passage. Where, however, by-passing is not provided for in the street system or is not necessary because of adequate thoroughfares in the down-town sections, through travel will have to be included in calculating street capacity. This rule does not work both ways, for obviously a street with capacity for no more than the through travel using it, could not be restricted to buildings of zero height, but, on the other hand, buildings on an extra capacious street cannot claim greater height allowance if the extra capacity of the street is best devoted to thoroughfare use. Where through travel occupies the greater part of a street at the expense of the local traffic and there is no other route feasible for it, accommodation of the local traffic by special means should be charged against the community. Certainly the abutting properties should not be penalized for the existence of traffic they have no part in creating.

There is a further point worth noting in relation to traffic in the down-town sections. One is accustomed to thinking in terms of intensively developed business sections which tend to draw traffic from all sides to a vortex where trouble is almost inevitable. When the interurban highways also pass through this point, as they generally do, the heightening of the trouble due to the through traffic is very noticeable. Under the operation of a really restrictive zoning ordinance, however, this peak of down-town development would necessarily be flattened out, more territory being covered and hence served by more streets. The accompanying decentralization of traffic destined for the district would automatically relieve the worst situations.

It is obvious that such studies as those just suggested might produce many different figures for allowable height in different parts of a single district. The final step in setting bulk limitations is the averaging of the conditions so as to obtain uniformity of requirements within each zone classification. The width of the street, frequently used as the basis for building height limitations to preserve a given angle of light, is not an equally good basis for limitations in respect to traffic. A wide street might be largely devoted to through traffic with which the abutting buildings have no direct concern, a narrow street might be served by a subway line giving it a capacity for local traffic much greater than that of a larger street without the subway, and so on. Where pedestrian traffic is an important factor, the distribution of transit lines

counts for more than the street width. An absolute height limitation for the district is better from the traffic standpoint, therefore, than a multiple of street width, and except in the case of a very regular street layout, this absolute figure would have to be obtained by an intelligent averaging of the figures computed for different streets and blocks.

Although the question of bulk limitation principally concerns business sections of cities where the greatest number of traffic problems arise, similar considerations apply to other sections. Wholesaling and warehousing areas develop a heavy street traffic which bears a fairly definite relationship to the size of the buildings involved. Apartment-house districts create quotas of traffic, both automobile and pedestrian, which are directly proportional to the size of the apartments. The more open residence districts form the principal exception, as the use requirements automatically restrict bulk at a point where traffic considerations do not come into play.

Thus far, the restrictions imposed by zoning on building heights and area have been discussed in relation to their influence on the streets in the immediate vicinity of the buildings regulated. There is a broader effect applying generally over the whole city, which is so closely associated with the effects of use zoning that it can most conveniently be treated under that head. Actually, of course, the use and bulk provisions of zoning ordinances are not independent of one another, for they both apply to the same properties and are designed for the same purpose, that is, to promote orderly and convenient urban growth. The arbitrary separation of the two subjects in this paper was made merely for convenience in treatment.

City traffic is made up of units traveling from point to point within the city, between points outside and those within, or between outside points the connecting route of which lies through the city. The number of these units originating in any given locality depends on the nature and intensity of human activities in that locality, the direction in which they want to go from their point of origin is governed by the nature and distribution of the activities serving as destination, and the total number of units going in any one direction depends again on both the nature and the intensity of the activities toward which that direction leads.

The thoroughfare system is supposed to connect the various localities of a city with one another and with outside points as directly as possible and over connecting links in each case ample for the traffic wishing to use them. The minor streets are supposed to collect or distribute to their individual destinations within the various localities all the units of traffic using the thoroughfares. The more nearly a street system does this with expedition and convenience, the more nearly perfect it is. The more nearly the number and desired routings of the units of urban travel can be calculated in advance, the more perfect a street system can be designed to serve them. Intelligent zoning provides a definite basis for just such calculations. By designating the kinds of activities permissible in various parts of the city, it controls what might be called the "impetus" for traffic between them. Different kinds of activities produce different kinds of traffic, as well as different amounts. Zoning classifies the various localities or districts according to the activities to

be conducted in them, basing their designation on dominant present-day use, natural fitness, or the needs of the future community. It then fixes the classification so that an inharmonious use may not be made of part of a district to the detriment of the activities properly conducted in it. In so doing it fixes the basis for traffic calculations on the assumption of definite localities ultimately occupied by definite types of activity, the traffic requirements of which are known.

Many different kinds and sizes of traffic streams are in daily progress between different localities, but one type is sufficient for an example. Those traffic streams conducting the inhabitants from their homes to their places of work and back again are the most important with which large cities have to deal. Stated in terms of districts, these streams connect residence with industrial and residence with business. If all the business and industry is in one central area and the residential section occupies its periphery, the streams converge; if business is at the center, residence next, and industry in an outer circle, the flow of traffic is much more diversified, the converging streams being smaller because of the movement of industrial workers outward and less exclusively one-way because of the workers traversing the city to reach their destination.

Assuming the first example to be an unzoned community with a thoroughfare system suited to its highly centralized life, the location of a few prosperous industries on cheap land at the outskirts might alter the entire situation, congesting minor streets previously unused and robbing the widest thoroughfares of their excuse for existence. The community, without having a hand in the shaping of the new condition, is nevertheless expected to pay for the widening and cutting of streets through the area of newly created values tributary to the factories, and to carry the potential loss of wide streets elsewhere not used to economic advantage.

The traffic requirements of an acre of industrial plant are not those of an acre of private residences. The latter may hold six or eight families, not more than 25% of whom are ever likely to leave the premises at one time, and can be amply served by a 10-ft. lane. The industrial plant may employ three shifts of 500 men each, arriving and leaving at as nearly the same moment as possible, requiring space for trolleys and busses, and several hundred automobiles, in addition to such trucking as may appear at the same time. It is only fair that the community should know which of these conditions it must contend with in a given locality. Industrial districts particularly are likely to create peculiar traffic problems because of the tendency of individual plants to have only one or two entrances to the premises. Several thousand operatives employed over twenty or thirty acres may all be bunched at one point at starting and quitting time. Even if plant organization does not require this, plant layout, with its surrounding belt of storage yards, coal bunkers, and railroad tracks, often leaves room for little else.

When a zone plan is being considered in relation to the thoroughfare system of the entire community, there is much more room for interplay between the two than is the case with the down-town streets alone. The conditions are not so fixed and the existing street system need not be finally accepted as a deter-

mining factor in the zoning plan. If an outlying section platted for sparse residential use is adapted for industry, there is little to hinder the street changes needed to serve that use; or if it is decided to throw a mixed district wholly into an industrial or a residential class, adjustment of the street system accordingly should involve little difficulty, a residential classification probably requiring no change and an industrial district being able to stand the cost without unfairness or hardship.

The map of use districts, therefore, can be blocked out on the basis of more relevant and vital factors than the street system which happens to exist, and a renovated thoroughfare system established on the basis of the tentative zone map. The framing of the thoroughfare and transit system is then a fairly simple and logical process. The districts in which people live and in which they work are set down in definite localities and with definite boundaries. The ultimate number per acre of those who can live or work in the several districts under the provisions of the zoning act is computable. The potential routes for travel show on the city map, or, better, on the zoning data maps if well prepared. Analysis of the kinds and amounts of traffic likely to use the various routes will determine their adequacy; if found to be inadequate, the possibilities of widenings and the opening of relieving routes can be given the same analytical study.

Transit lines may be located in much the same way, and rapid transit routes fitted into a scheme of surface feeders. The Detroit Rapid Transit Commission has announced its unwillingness to decide on rapid transit routes and facilities until the communities to be served are zoned.

Some leeway may even be found to shift zone boundaries to throw needed traffic arteries away from congested points, and this possibility illustrates the need of simultaneous street and zone system study. When lateral expansion of the down-town district is required because of the impossibility of further vertical expansion without drastic street changes, it is logical and justifiable to direct that expansion where a good street system exists or can most easily be provided.

It is even within the power of the zoner to encourage decentralization of city activities in the interest, among other things, of better traffic conditions. The spread of the business district necessitated by height restriction can be used to encourage satellite business sections strategically placed, and the assured character of the surrounding neighborhood under zoning will aid in their establishment and growth.

To sum up, zoning helps the traffic situation by showing where people are going to live and how many are going to live there, and where people work and how many are going to work there. If a section of the street system is already well established through the development of the surrounding district, the further growth of the area dependent on those streets is checked before reaching the point where they must fail to function. In places where expansion or change in the street system is possible, zoning provides the basis for determining the extent and nature of the changes needed. The chaotic growth characteristic of American cities largely nullifies city-planning efforts. Zoning by marshalling the factors of growth under an orderly control, makes scientific city planning possible and worth while.

THE INFLUENCE OF ZONING ON THE DESIGN OF A TELEPHONE PLANT

By Albert P. Allen,* Esq.

In many ways a better title for this paper would be "The Design of the Telephone Plant and Its Effect upon the Preparation of Zoning Ordinances". In suggesting this reversal of the title, it must not be assumed that the telephone interests are in any way antagonistic to the plan (which has met with so much favor in recent years) of systematic zoning of large cities in order that their future growth may be controlled and regulated along lines which contemplate, and undoubtedly will secure, the best results for the citizens as a whole. The telephone company, necessarily, is a part of the city in which it operates and is dependent on that city's successful growth. Therefore, anything which is for the ultimate benefit of the city must be of benefit to the telephone company.

In addition, however, to this general interest which the telephone company takes in common with other good citizens, it has a second and even stronger reason for being favorable to the establishment and maintenance of a good zoning ordinance in each city. Such ordinances tend to stabilize and make more dependable the development in the various sections of a city, and, therefore, tend to make easier and more reliable its estimates of future growth and plant requirements in such sections.

Therefore, the real reason, the proposed reversal of title might be considered an improvement, from the point of view of the telephone company, is merely that the design of the telephone plant in any city is almost wholly controlled by the service demands of the public; and unless the zoning ordinance (and, incidentally, all other ordinances) enforced in any city, is drawn in accord with those service demands, any changes in the design of the telephone plant which are caused by such ordinances, although opposed to the facts of the business, can be shown to be harmful rather than beneficial to the public as a whole.

The true interrelationship between the zoning of cities and the carrying on of the telephone business within them can be understood only by those who not only appreciate the real relationship of the present-day public service corporation to the public it serves, but also understand how and why a modern telephone plant is built as it is. These questions are both large subjects, and, in this short paper, the writer can do little more than call attention to some of the basic facts, without going very deeply into the supporting arguments and explanations regarding them.

The first important fact is that the telephone company, being publicly regulated as to the rates it may charge, can have no selfish or mercenary motive in either supporting, opposing, or criticizing any given zoning law or regulation. Its position necessarily is a neutral and professional one, of

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interest in efficiency and economy for the ultimate benefit of its patrons, the public. It is not the stockholders, but the customers, of a telephone company who should be most interested that no unnecessary costs of providing service are imposed on the company, because it is they, and not the stockholders or employees, who suffer from, and have to pay for, such costs. This is the first point which must be appreciated by those who have the duty, or privilege, of establishing zoning, as well as all other, regulations affecting the utility services.

The second important fact affecting telephone companies is that the proper and economical plan on which they must build is controlled within rather narrow lines by their customers and by the facts of the business, and not merely, as one might assume, by the whims or theories of their engineers. Each central office or exchange building has to meet several rigid conditions, all controlled by the present and future number and distribution of the customers to be served therefrom, as follows:

1.—It must be within economical transmission distance of all the subscribers it serves.

2.—It must serve the total number of subscribers to be located within such economical radial distance.

3.—Each subscriber must be directly connected by a pair of wires (on an individual or party-line basis) with the switchboard at such a central office.

4.—All such central-office switchboards in a city must be interconnected, by suitable inter-office trunks, for direct and instantaneous connection on local calls, and for prompt connection to all toll or "long distance" points outside that city or exchange area.

5.—Each central office must be located at the economical "wire center" (or telephonic "center of gravity,") of the area it serves, with due allowance for the effect of the trunk circuits and cables on the final determination of such economical centers of distribution.

6.—In small cities, which can be served by a single wire center and office, the center often falls in or near the business district.

In large multi-office exchanges, however, many sub-areas serve residence stations almost exclusively. Moreover, the densest development often occurs in the better neighborhoods, in which case the economical wire center may be determined as lying well within some highly restricted zone.

7.—To move a central office far from its natural economical wire center adds greatly to the investment in conduit and cables and to the corresponding costs of giving service, since, as previously stated, all subscribers' lines must be carried into the central office.

8.—Telephone central offices should be located in clean, quiet neighborhoods, free from the noise, smoke, and dirt of steam railroads, street cars, or heavy street traffic.

It will be noticed that all the eight conditions mentioned have to do with the location or size of the telephone central office, and that none of them has to do with the other features of the design of the physical plant. Such features as the use of underground or aerial construction, system of distributing wires

to reach the premises of individual subscribers, type of switchboard, and other matters which are usually thought of in connection with a telephone plant, are in no way controlled by the regulations of a zoning ordinance. Therefore, it may be said, in general, that this discussion involves only the location and size of central office buildings which are called for by an efficient and economical design of the telephone plant as a whole. Obviously, such a design must be interfered with as little as possible by the conditions of a zoning ordinance if the telephone service is to be rendered in the most economical manner and at the lowest possible rates.

Moreover, the telephone company would not raise these points if their effect on the cost of service was not known to be great enough to be a controlling factor in the determination of necessary rates for service; and, especially, if the differences in cost would be eliminated, instead of being accentuated, by growth, as time goes on. In several cases, if the zoning ordinances had been passed as originally drawn, and had not been changed at the suggestion of the telephone company, the cost of telephone buildings and cable plant would have been greatly increased.

The various conditions which have been listed as controlling the proper location and size of telephone central offices, will now be discussed a little more in detail. These conditions should be recognized as important and necessary in the zoning ordinance itself, and by any Board of Appeals which may be created under such an ordinance.

1.—A Central Office Must Be Within Economical Transmission Distance of All the Subscribers It Serves.—By "transmission distance" is meant the combination of length of circuit and size of conductors which apply to the lines of the various subscribers served by any one central office. Although it is true that the state of the art to-day will enable telephone engineers to produce proper telephone transmission over almost any length of circuit, nevertheless, it is obvious that there is a practical distance from the central office battery supply beyond which it is uneconomical to attempt to serve subscribers in any exchange, due to the constantly increasing quantities of copper that would be needed, or the reactions on transmission that would result if those large quantities are not provided.

There is no uniform or fixed maximum distance which is applied even to common battery subscribers, because the cost varies with the number of circuits in a cable as well as with the length and size of the conductor used to supply any given section. Moreover, the strict maintenance of an inflexible maximum distance would create uneconomical fractional office areas in any large city. For all practical purposes, however, this first condition means that the distance which any subscriber can be located from the central office is quite limited, and this condition creates the necessity for a large number of central office buildings throughout any large metropolitan area. Moreover, it means that the location and size of the various areas are fixed by the service demands of subscribers and not in any arbitrary way by the telephone company.

2.—A Central Office Must Serve the Total Number of Subscribers to Be Located Within Its Economic Radial Distance.—This condition will probably

be recognized by all engineers as a necessary corollary to Condition 1. It forms the basis for the economic necessity for the ultimate size and height of the telephone building which must be permitted.

3.—Each Subscriber Must Be Directly Connected with the Switchboard Located at Such a Central Office.—This also states one of the obvious conditions of modern telephone service, and it affects both the location and the size of the central office building, when considered in connection with Condition 4. It illustrates the particular feature which perhaps chiefly distinguishes the distribution of telephone service, if that term can be used, from the distribution of other utility services, either transportation, electric light, electric power, gas, or water, the difference being that each telephone subscriber has to have his own individual connection with the source of service supply, whereas all other utilities serve their customers by common mains, or facilities used in common by large groups of patrons. However, they, too, have important problems to be considered in connection with zoning.

4.—All Central Office Switchboards Must Be Interconnected by Suitable Inter-Office Trunks.—The reason for this condition is that all central office switchboards must be available for complete inter-office local service; and also, of course, that they be made available for toll service throughout the telephone system. Thus, a subscriber in Detroit, Mich., has his own connection to his local switchboard, and by means of a trunk to "long distance," a toll line to New York, N. Y., a trunk from the toll-board in New York to a local central office there, and connection with the circuit connecting a New York subscriber with that central office, he can converse with the subscriber in New York with a minimum possibility of interference from any other subscriber. All parts of such a system of interstate telephone connection are so balanced in regard to transmission standards as to give the greatest economy of plant costs for the combined system as a whole.

The exact method of obtaining this balance of transmission requirements between the various parts of the system is quite technical, and would require a extended paper for its complete explanation. For the purposes of this discussion, therefore, it must be taken for granted that the economical balance between these factors has been fairly well determined and that any conditions arbitrarily imposed on the telephone company, which would upset that balance, would be very unfortunate to the users of telephone service because of the increased costs which would result therefrom.

5.—Each Central Office Must Be Located at the Wire Center of the Area It Serves.—The wire center, or location of the central office, is the telephonic center of gravity of that part of the city which it serves. Obviously, it is not always, if ever, the exact geographic center of that area, being moved away from such a center by a preponderance of either subscribers' circuits, or more expensive but fewer trunk circuits lying in one direction as compared with the opposite direction. It is a center, therefore, which is determined by a study of the present and ultimate number and distribution of subscribers. The actual location of the central office building cannot be moved far from the theoretical center without a large increase in the cost of service.

The extra cost due to such a move of the actual from the theoretical wire center is especially costly in the case of a central office which has already been planned; which will be put into service in a few years; and which, therefore, has had its effect on the underground conduit, pole line, and cable work already completed in accordance with such fundamental plan.

In the same way it would be very expensive to change the ultimate capacity of a building already in service by preventing, through a zoning ordinance, for instance, the completion of the building to its contemplated height or size, thus making it impossible to fulfill Condition 2, which specified that it must serve the total number of subscribers located within the proper radial distance from it. The only alternative, as can easily be seen, is to create an additional, uneconomical, inefficient, and costly central office to serve those subscribers thus ruled out of their natural wire center.

6.—In Large Multi-Office Exchanges Many Sub-Areas Serve Residence Stations Almost Exclusively.—This fact makes it impossible, without prohibitive cost, for the telephone company to serve residential areas without placing some telephone offices within and near the center of such areas. The telephone company much prefers a location which is acceptable rather than objectionable to the residents in the immediate neighborhood. It can be trusted, therefore, to select locations satisfactory to the public whenever this is possible without adding too greatly to the total cost of the service. Moreover, the Bell System and, the speaker believes, practically all telephone companies have made it a practice for a great many years to employ the best architects and to construct a type of building for use in residential areas which is attractive in itself and which conforms as fully as possible to the surroundings in which it is to be placed. In many instances, as a matter of fact, a telephone building is an admitted benefit to the neighborhood and, architecturally, is as unobjectionable as a public library, a school, or an art museum which are often located in residential sections.

7.—To Move a Central Office Far from Its Natural Wire Center Adds Greatly to the Investment in Conduit and Cables.—The reasons for this increased cost have been clearly indicated; and in any instance which might arise under a zoning ordinance, the local engineers could calculate closely just what the expense of any proposed change in plans would mean, both at the present time and ten or fifteen years hence, which is often more important. The calculations would show that any move of a central office from its natural wire center will add greatly to the investment in conduit and cable.

8.—Telephone Central Offices Should Be Kept as Free as Possible from the Dirt and Smoke of Steam Railroads, Street Cars, or Heavy Traffic.—Dirt and smoke are always objectionable to an operating force, and are also particularly harmful to the delicate apparatus in the central office equipment of a telephone company. Noise is also objectionable in an operating room, and even the operators are taught to speak in low tones in order not to disturb adjacent operators and not to create an air of confusion in the operating room. With the introduction of machine-switching service, the objection to smoke and dirt is somewhat increased; but even then the objection to noise remains, because

the human element is never eliminated from the central office, even when most of the regular connections are completed by machinery.

Therefore, any provision of zoning which would assume that a telephone office might just as well, if not more logically, be located on a business street or in a district reserved for ordinary industrial uses, rather than on a quiet residential street, involves objections to such a procedure over and above the direct cost which it might add to the giving of service.

In addition to the statements just given a few points will be mentioned, that should be kept in mind when a zoning ordinance is being drafted. They are offered for the protection of the telephone subscribers against unreasonable and unnecessary additional costs for service, which will result if they are overlooked:

1.—The ordinance should permit the erection of telephone buildings in any use district, in order that they may be located at the wire centers.

2.—The ordinance should permit the erection of telephone buildings to economical heights. For example, if the height limit in residential districts would not permit of an economically designed building, provision should be made whereby exemption could be obtained to permit the erection of the building to a satisfactory height. In this connection, it must be remembered that each floor of a modern telephone building requires about 13 ft. of height.

3.—The ordinance should also permit a reasonable use of the plot on which the building is to be located. Some ordinances restrict to a very low percentage, in certain areas, the area of lot which may be occupied by the building. This may constitute an unnecessary and unreasonable restriction if all the facts about the erection of the usual telephone building are considered.

4.—Provision should be made to permit the completion of buildings constructed prior to the adoption of an ordinance to less than the height or bulk to which they were originally designed. This is an extremely important provision; and certainly no objection should be made to the completion of a telephone building along the lines originally planned, in order to give efficient and economical service to the subscribers.

5.—Some ordinances contain a provision that a building may not be occupied in any part for any purpose until the building is entirely completed. This might be a serious restriction on many desirable activities, and it is important that the ordinance provides for a temporary certificate of partial occupancy in advance of the final completion of a building.

6.—A Board of Appeals should be created to hear appeals from the decision of the administrative officer. In other words, there should be a body to which the telephone company may go for relief in case the strict terms of the ordinance would impose an unnecessary or unjust handicap on the telephone service. The experience of zoning experts indicates that a Board of Appeals is most necessary in order that the ordinance may be reasonably applied so as best to promote the public welfare.

7.—It is most important that suitable State enabling acts be in force, which make mandatory certain provisions in zoning ordinances enacted under them, compelling proper treatment in any case in connection with which, from the viewpoint of public convenience or welfare, reasonable necessity for any

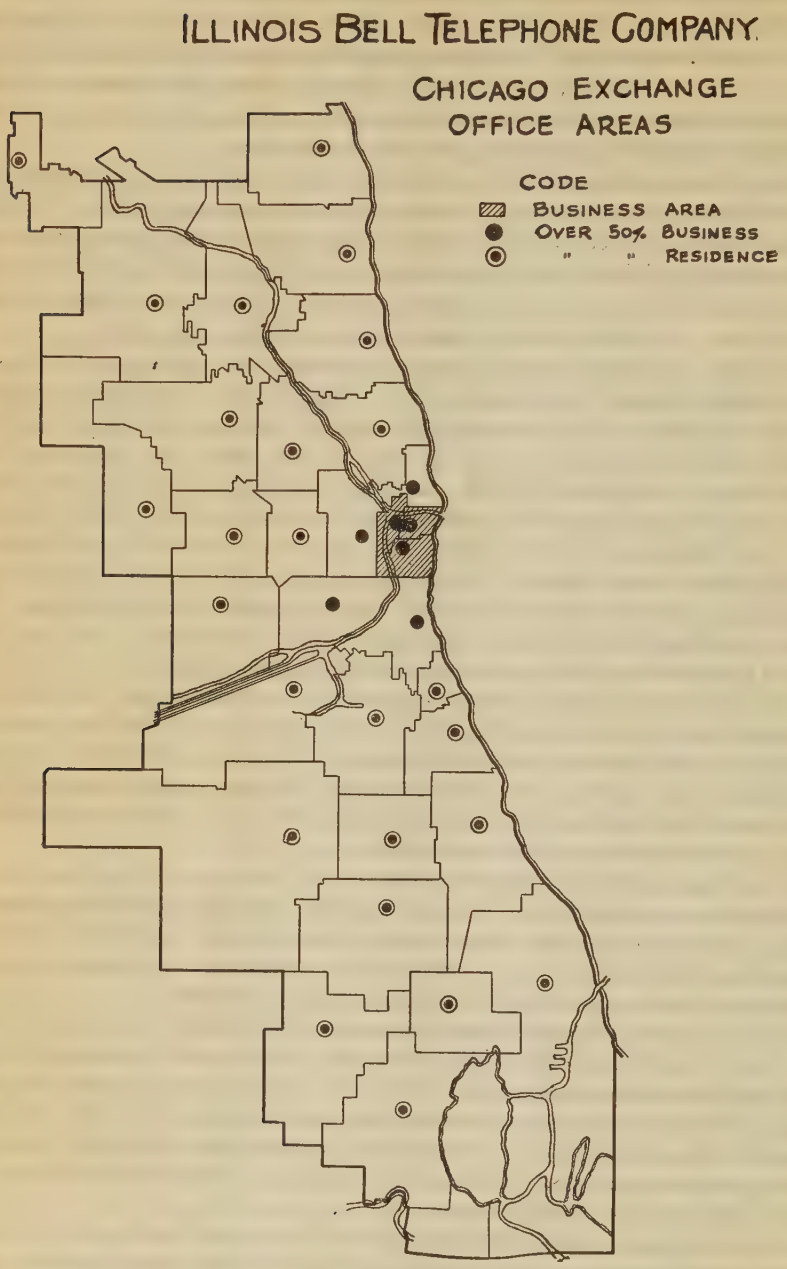

ILLINOIS BELL TELEPHONE COMPANY
CHICAGO EXCHANGE
OFFICE AREAS
CODE
BUSINESS AREA
OVER 50% BUSINESS
" " RESIDENCE

FIG. 3.

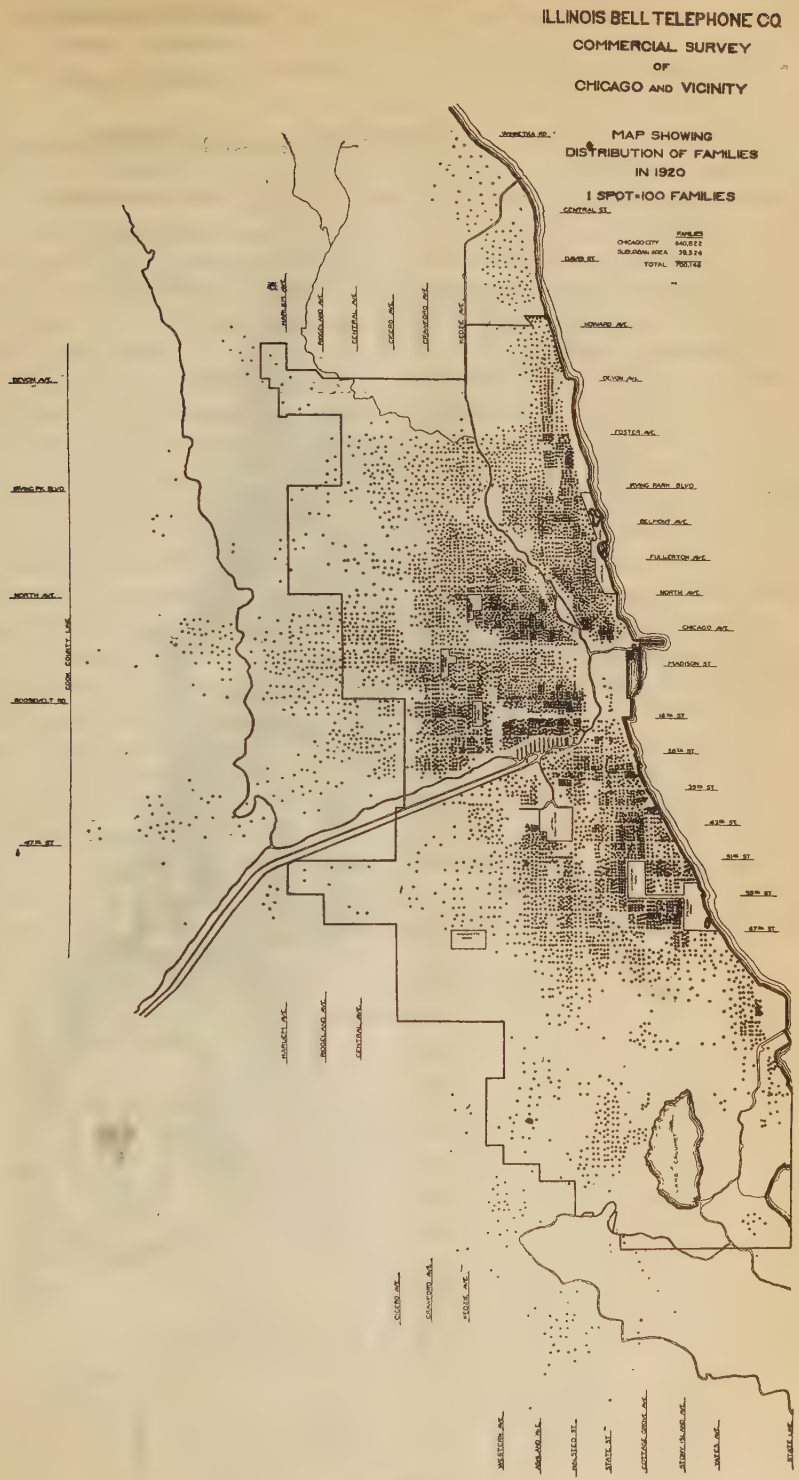

ILLINOIS BELL TELEPHONE CO.
COMMERCIAL SURVEY
OF
CHICAGO AND VICINITY
MAP SHOWING
DISTRIBUTION OF FAMILIES
IN 1920
1 SPOT=100 FAMILIES
FAMILIES
CHICAGO CITY 640,872
SUBURBAN AREA 38,324
TOTAL 700,748
CENTRAL ST.
DAVIS ST.
HOWARD AVE.
DEVON AVE.
FOSTER AVE.
IRVING PARK BLVD.
BELMONT AVE.
FULLERTON AVE.
NORTH AVE.
CHICAGO AVE.
MADISON ST.
16TH ST.
26TH ST.
35TH ST.
43RD ST.
51ST ST.
59TH ST.
67TH ST.
WILMETTA RD.
DEVON AVE.
IRVING PK. BLVD.
NORTH AVE.
ROOSEVELT RD.
67TH ST.
COOK COUNTY LINE
HARLEM AVE.
RIDGELAND AVE.
CENTRAL AVE.
CICERO AVE.
CRAWFORD AVE.
KEDZIE AVE.
CICERO AVE.
CRAWFORD AVE.
KEDZIE AVE.
WESTERN AVE.
ASHLAND AVE.
ARCHER AVE.
STATE ST.
COTTAGE GROVE AVE.
STONY ISLAND AVE.
YATES AVE.
STATE AVE.

FIG. 4.

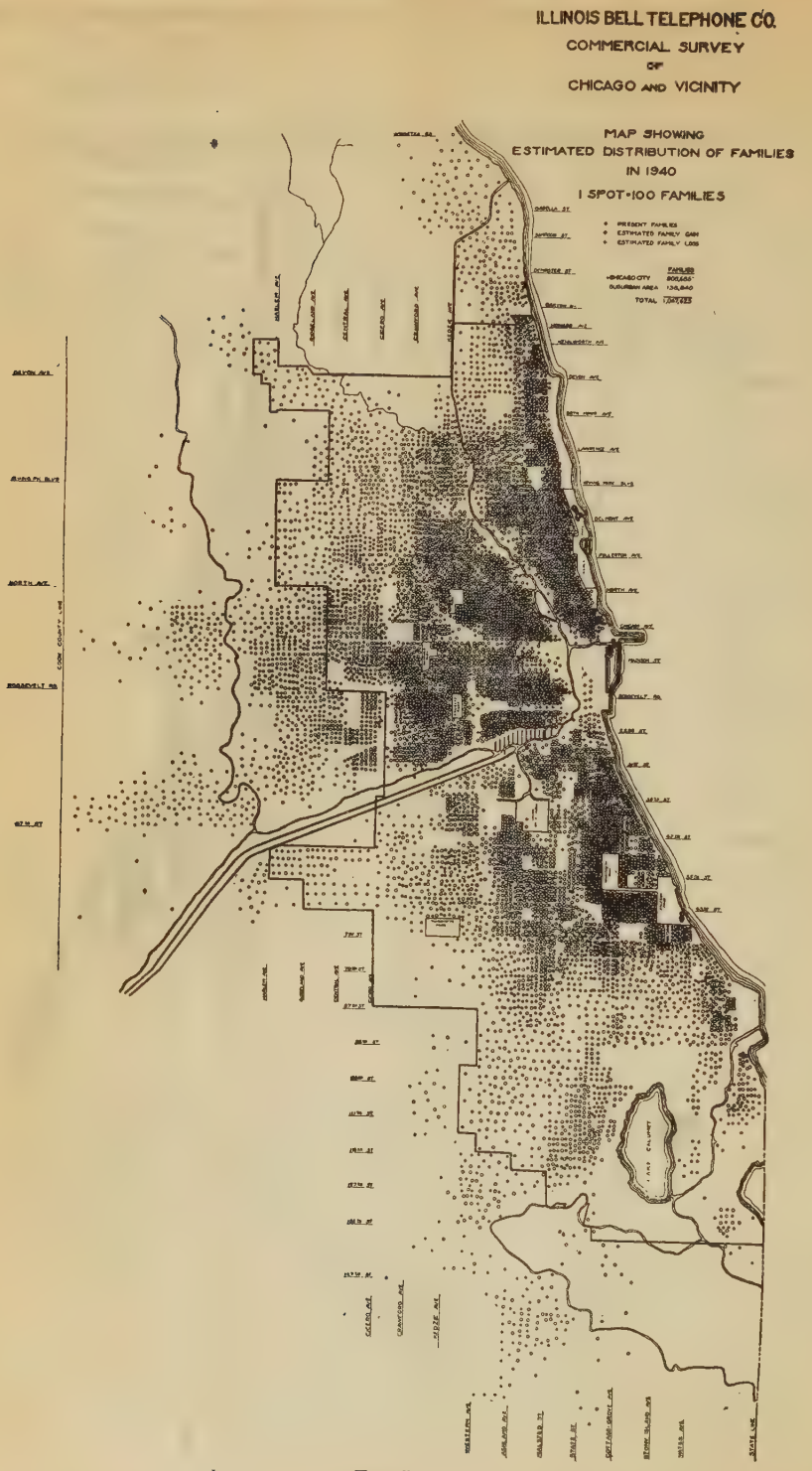

ILLINOIS BELL TELEPHONE CO.
COMMERCIAL SURVEY
OF
CHICAGO AND VICINITY
MAP SHOWING
ESTIMATED DISTRIBUTION OF FAMILIES
IN 1940
1 SPOT=100 FAMILIES
PRESENT FAMILIES
ESTIMATED FAMILY GAIN
ESTIMATED FAMILY LOSS
FAMILIES
CHICAGO CITY 806,550
SUBURBAN AREA 134,840
TOTAL 1,941,390

FIG. 5.

project can be proved to the satisfaction of the Board of Appeals, or Adjustment.

Telephone buildings are intended primarily for the housing of delicate apparatus and the handling of conversations between subscribers, and they will be limited to such use by the telephone company wherever the occasional concurrent use of the same building, or lot, for storehouse, garage, maintenance headquarters, or even commercial offices, is considered unnecessary and objectionable at any particular location. As a matter of fact, such extra use is seldom desired by the telephone company in a residential district; and if such a question ever arose, the company would be willing to have it decided by a Board of Appeals. It desires in every way to be guided as far as practicable by the general provisions of any zoning ordinance adopted for the benefit of any city in which it operates, and its possible objection to any such ordinance will always be restricted to such fundamental service and cost items as, in the opinion of its experts, will unquestionably lead either to the curtailment of its service or to a noticeable increase in the cost thereof. It would appear, however, that provisions can be made in zoning ordinances, which should reasonably provide for the telephone problem and, at the same time, be in conformity with the broad fundamental principles on which zoning is understood to be based.

In conclusion, and to illustrate some of the conditions described, the present Telephone Exchange, in Chicago, Ill., as a whole is divided into 32 office areas of which only 7 serve more business than residence stations, 3 of these being in the congested Loop area and the other 4 adjacent thereto. The other 25 offices serve chiefly residence subscribers, as indicated by Fig. 3. Figs. 4 and 5 show, by spot maps (on which each spot or dot represents 100 families), the number and distribution of families in and near Chicago in 1920 and the number and distribution of families which studies indicate for 1940. From Figs. 4 and 5 it seems evident that it will be impossible to locate 40 central office buildings in Chicago in 1940 and not have many of them in purely residential districts.

THE INFLUENCE OF ZONING ON THE DESIGN OF PUBLIC RECREATION FACILITIES

By C. E. Brewer,* Esq.

Zoning is an undertaking to create a complete and satisfactory living environment for the inhabitants of a city or community. It is an attempt to bring into a community every facility which is vital and necessary to the welfare and happiness of the citizens, and yet not spoil the community as a good place in which to have a home. Transportation, sanitation, streets, water, light, and recreation are necessary facilities to any community. How to have them without injuring the value of property is a problem confronting all city planners and city engineers. Badly planned cities, resulting in narrow and crooked streets, poor housing conditions, a lack of sewers and drainage, have caused more illness, discontent, and loss of life than will ever be known.

Work, play, and rest fill the twenty-four hours of each man's life. Where and how a person works is the concern of the progressive business man, as it means profit to him. What a workman does after 7:00 A. M. depends on what he does after 5:00 P. M. of the previous day. The kind of homes is also the concern of every one in the community, for bad living conditions re-act on the health and physical efficiency of the working man. The place and the mode of recreation are the vital problems of every man, woman, and child. Indirectly, recreation affects a vital part of a community's life. Play is an absolute requirement for efficient and continued existence. It is a mental and physical relief from any exhausting activity or environment. It substitutes clean, wholesome use of leisure time from the drudgery of arduous and confining work and for the repressed inherent instinct which one has. Every one has inherent instinctive desires for play. They are generally expressed in creative play. Due to the highly specialized form of industry, in which a man works on a single operation and does not bring the product to a finished condition and has no opportunity to give vent to these inherent creative desires, they must be given expression during his leisure or play time. Therefore, playgrounds and playfields should provide an opportunity to escape from the walls of brick and mortar of the factory, or the dull, toneless environment of the neighborhood. Unfortunately, in most cities too little attention has been paid to the facilities for recreation.

The first attempts to provide open spaces, or "breathing spots", were the public parks. The public park with the "Don't walk on the grass" sign, with its formal flower beds and clusters of shrubbery, did not afford an opportunity for people to give vent to their pent-up and repressed desires for play. Consequently, playgrounds and playfields with large open spaces for all forms of competitive games were developed in modern cities to meet the demand which the public park did not fulfill.

* Commr. of Recreation, Dept. of Recreation, Detroit, Mich.

Parks require large tracts of land for development into formal and artificial beauty spots, whereas a playground requires a small area in comparison. The amount of land required for parks in comparison with that of playgrounds has had an important influence on zoning, particularly if the city has grown rapidly and all the available land was occupied before any thought was given to a plan of acquiring playgrounds.

In zoning, much consideration must be given to the size and development of the playground and playfield. Playgrounds are differentiated from playfields according to the use and kind of activity organized on them. A playground is a small tract of land of from 1 to 3 acres on which playground apparatus is placed and games requiring no special skill are played, a play director being in charge. A playfield is a larger tract of land on which games of skill, such as baseball, football, tennis, soccer, etc., are played.

Zoning has a great influence on the distribution of playgrounds and playfields throughout the city. If the district to be zoned permits business places and factories, and if the tendency in the future of the district is toward more commercial places, then playgrounds should be smaller and situated farther apart. They should be designed for use by the employees of the neighboring factories during and after working hours, and should be situated so that they can be readily sold when they are no longer needed by the city as recreation places.

In districts reserved for residential purposes, the distances apart of the playgrounds should be comparatively short, especially if there are heavily trafficked streets through the district. The various playgrounds in such a district should be of a size to serve best a distinct function, whether it is a small children's playground, a playground for adults, or for both. They should be located throughout the community so that they are easily accessible to those intending to use them. The prevalent impression is that playgrounds should be within ½ mile of each other. A survey recently made in Detroit, Mich., proves the fallacy of this argument. In this survey, an accurate check on the distance, with 21 347 children attending the playgrounds in 1 day, shows that the majority travel less than ¼ mile. Table 1 gives the results of the check made in Detroit.

Assuming an average of 10 city blocks per mile, it will be noted that, to be effective, playgrounds should be not more than 2 or 3 blocks apart. If they are placed at greater distances, more than 65% of the children will not attend. This is largely due to parents refusing to permit children to go that far from home to play, especially if it is necessary for them to cross streets with heavy automobile traffic or street cars. With the increasing flow of people from the rural districts to the city, the time is not far distant when it will be necessary to have a playground in every city block, particularly if the trend of building large apartment houses, instead of single homes, increases in the large cities.

Equipment on the playground, the area of free play space, whether tennis courts are desired, and space for other games, must be seriously considered when the boundaries of any playground are fixed. In fact, no playground

should be acquired until the needs of the community and the number of children and adults that would make use of it are ascertained. The plan of development should be definitely settled before the acquisition of any playground is made.

TABLE 1.

ATTENDANCE.

Distance, in blocks.	Number.	Percentage.
1	7 199	33.7
2	3 994	18.7
3	3 146	14.7
4	1 889	8.8
5	1 418	6.6
6	897	4.2
7	629	3
8	455	2
More than 8	1 720	8.3

In zoning a community, the topographical features must be taken into consideration in planning a playground. One cannot mathematically place one playground here and another half a mile away. Canals, rivers, railroads, and heavily trafficked streets must be taken into consideration. The proximity to factory and commercial houses must be considered by the city planner. It would be foolhardy to place a playground in a community where the children had grown up and married and had left the home for homes of their own and only the old folks remain. It would be equally as bad not to place a sufficient number of playgrounds in a new district, especially where the child population is very dense.

Table 2 shows the distance that children of various ages travel in attending the playground. These tabulations, totaling more than 25 000 children, were taken in Milwaukee, Wis., St. Paul and Minneapolis, Minn., and Detroit.

From Table 2 it will be noted that, in the average city, 50.4% of the children travel ¼ mile or less; 74.5% travel ½ mile or less; 86% travel less than ¾ mile, leaving only 14% who travel 1 mile or more to a playground. This table also shows that 74.5% of the children in these four large cities travel less than ½ mile to a playground.

It is significant that in St. Paul and Minneapolis less than 35% of the children travel ¼ mile or less to a playground, whereas in Detroit, 52.63% travel less than 2 or 3 blocks. This shows that in the larger cities where street traffic is heavy and where the population is increasing rapidly, the playgrounds must be closer together than has been the custom in the past.

In order to provide growing cities with proper playground facilities in the future, the writer believes that any zoning ordinance should require every new subdivision to have at least a 2-acre playground for every 160 acres of land subdivided. Whether this can be legally done must be determined by the Courts through the proper channels. However, it is believed that every realtor can be educated and persuaded that the setting aside of such a small strip of land is in reality a profitable move on his part. People grown wise in the

ways of the city are now demanding that playgrounds be provided for their children and any realtor can sell his property more readily and at a better price if the purchaser knows that a playground will be within a short distance from his home.

TABLE 2.

Age of child, in years.	¼ Mile.		½ Mile.		¾ Mile.		1 Mile.		More than 1 Mile.	
	Number.	Percentage.	Number.	Percentage.	Number.	Percentage.	Number.	Percentage.	Number.	Percentage.
Less than 3	952	3.8	190	0.76	88	0.35	26	0.10	37	0.15
3 to 7	2 257	9.0	809	3.23	287	1.15	126	0.5	99	0.39
7 to 12	4 802	19.2	2 300	9.2	1 110	4.44	600	2.4	463	1.85
12 to 15	2 126	8.5	1 179	4.71	654	2.61	274	1.09	398	1.59
More than 15	2 461	9.84	1 548	6.19	772	3.08	476	1.9	966	3.86
Total	12 598	50.4	6 026	24.1	2 911	11.65	1 502	6.0	1 963	7.85

Zoning should demand that all playgrounds be fenced, not only because fences improve their appearance and that of the abutting property, but because it is a "safety first" measure. Children are not so likely to run heedlessly off the playground under the wheels of traffic if a fence is there to stop them. Playground fences should be constructed with galvanized steel posts and steel wire, heavily galvanized after fabrication. A fence, 5 to 7 ft. high, has been found to be the most satisfactory. The fencing of playgrounds greatly reduces the problem of landscaping as flowering vines, rambler roses, and other climbing species of plants along the fence, greatly improve the playground.

In landscaping a playground, the element of beauty must not be overlooked, yet as much free play space as possible must be preserved. Flower beds or shrubbery beds have no place on the playground, except in the corners of the fence. They would occupy too much valuable space and require an overproportionate amount of time and labor in maintenance. If shrubs are desired as screens, the fences should be placed back from the sidewalk and the shrubbery planted outside the fence, but not close enough to the sidewalk to interfere with pedestrian traffic. Trees should be planted near the fence or near the apparatus in order to afford as much shade as possible. Trees of low height that will provide a dense shade as quickly as possible should be planted in preference to slow-growing elms.

Surfacing is a big problem. It is difficult to make the grass grow when it is being constantly trampled down, yet a bare playground is unattractive to the children and dust arises, causing much nuisance to the housewife. The proper surfacing for a playground has been the source of much discussion. Cinders are dirty, black, and stifling. Gravel is unattractive, and the stones are too much of an attraction to the small boy who desires to throw them. Tan-bark has not proved satisfactory. Some cities have climates which make it neces-

sary to have surfaces which, in winter, must withstand the constant freezing and thawing. Others require an easy drainage. Some cannot have surfaces which absorb too much heat. Others, because of financial reasons, must have a cheap surface.

No surfacing has yet been found which proves satisfactory under all conditions for every playground. A playground surface must be suitable, that is, porous enough for drainage, yet firm enough to stand up under constant use when it is wet and not raise a dust when it is dry. In Detroit, a surface of limestone screenings and calcium chloride has proved very satisfactory. It is porous, drains easily, can be used soon after a rain, and does not develop soft spots. It is soft in thawing weather, yet packs down readily when the frost is out of the ground. Calcium chloride keeps down the dust and also forms a binder coat to keep the surface firm. This kind of playground surface has a foundation of 4 in. of hard cinders, wet down and rolled (the cinders are kept wet while being rolled), then 3 in. of limestone screenings is next applied. This is wet down and rolled to a true grade. Next, calcium chloride is applied about 1½ lb. per sq. ft. (calcium chloride absorbs moisture from the air and keeps down the dust), then ⅛ in. of sharp torpedo sand is applied. The calcium chloride in forming a binding coat with the limestone screenings and the sand gives a firm, smooth, white finish to the playground.

Buildings, especially lavatories and storage space, are required on each playground. The design of such buildings should be equal to or better than that of the surrounding architecture. The buildings should be permanent and of brick or stone. Wooden buildings should not be erected on parks or playgrounds as they soon become an eyesore rather than a benefit to the community. They are also expensive to maintain, as they soon show the effects of the wear and tear and abuse which, unfortunately, the American public gives to its public buildings.

Playground apparatus should be made of galvanized steel pipe, guaranteed against rust. Rusty apparatus frames, and usually those made of wood, are unsightly. The following pieces of apparatus have been considered the minimum necessary for an efficient conduct of a playground: Two sets of steel swings (three swings in each set), one set for boys and the other for girls; one set of kindergarten swings for babies under three years of age; a horizontal ladder for girls; a giant stride for boys; a 16-ft. coaster, or wave slide; a sand-box, 4 by 8 ft.; and four teeter-totters.

In summarizing, it has been shown first that zoning does affect the distribution and location of playgrounds, the fencing, landscaping, and architecture of buildings; and, second, that the city planner must ascertain the needs of the community and the density of child population in the community, and determine the number of people that would probably use the playground, before any attempt is made to locate a playground in a given district or zone.

THE INFLUENCE OF ZONING ON THE DESIGN OF TRANSPORTATION SERVICES

By J. Rowland Bibbins,* Esq.

The assigned subject of this paper would apparently confine it largely to rail movement, but it is now impossible to avoid consideration of all surface transport on the thoroughfares of a city. Transportation must be conceived as covering all carriers used in public service. Traffic is simply the by-product, whether *via* rail, water, or "rubber". It is a composite problem, unfortunately not widely appreciated as such, and the street plan is of fundamental importance to local transport, as private right of way is limited to railroads, rapid transit, or grade-separated trunk highways—all very scarce in American cities.

What Is Zoning?—Zoning has been defined, as follows:

"A fair, common-sense regulation governing the use of private real estate." (Hoover Advisory Committee on Zoning.)

"The legal exercise of State police power * * * an amplification of welfare and safety laws essential to orderly city growth * * * broad enough to meet ever-changing conditions. * * * As the primary purpose of City Planning and Zoning is to regulate and distribute the population of a city * * * the safest way to accomplish this is by *Density* Regulation." (Bartholomew.)

"Based upon a frank acceptance of existing conditions * * * also upon judgment as to future growth and requirements * * * the transport system is the chief factor in determining business and residence building." (Whitten.)

"After a scheme of streets, transit, transportation and recreation has been determined upon, it becomes a matter of evident reasonableness to regulate property uses in all parts of the city in accordance with these several elements." (Bartholomew.)

Which Comes First—Zoning, Transportation, or City Plan?—Obviously, these subjects should not be dissociated, yet this frequently happens. The "Municipal Index" cites 200 zoning precedents to-day, but only 75 "comprehensive" city plans:

"Other planning features such as * * * port-development, routing of transit lines, have found place in many planning reports, but the combination of streets, zoning, and recreation into a well-ordered whole is essential to guide the building of the future city."

The writer does not fully share this relative appraisement of the value of control factors, especially of transportation, so often neglected. There is a suggestion of unbalancing here, running against economic laws. For basic transportation plans have all too rarely been studied in sufficient technical perspective or even deeply considered. A wealth of such data should be had from these 200 zoned cities.

A natural fixation arises from topography, geography, hydrography, railroads, prevailing winds, etc. Business is automatically fixed by transportation

* Cons. Engr., Washington, D. C.

routes. These elements cannot be divorced without assuring adequate facilities elsewhere. The defect in the usual city program is that, although zoning control is established, the needed transport facilities are not provided, and rarely planned as a vital part of the program, otherwise destined to defeat or delay.

The Traffic Situation To-Day.—All large cities lay claim to the doubtful distinction of being "congested". How many have sought the basic reasons why? How many are using present facilities to full advantage? In a recent survey the writer found an average efficiency of use of 36 outlet streets to be only 25% of that possible, even assuming solid parking, that is, about 300 vehicles per traffic lane per hour. Good practice should be 1 000 vehicles per hour and one street in the country is to-day carrying 3 000. This survey also showed a daily commercial vehicular movement of one-half that of automobiles and the latter carrying a rush traffic equal to that of the transit system, largely traceable to defective street system. This city is zoned. It is found in Chicago, Ill., that 60% of the store deliveries are made at the curb and that 30% of the street movement is due to "milling" for position. There is 1 vehicle movement down town for every 2 automobiles in the district. The writer found the ratio in Indianapolis, Ind., to be 1 to 2.5, or less. In New York, N. Y., 70% of the north-south traffic is concentrated, from Fourth Avenue to Eighth Avenue.

The Motor's Growth.—The writer's studies of the increase in motor vehicles have indicated an amazing fact—a uniform rate of absorption for the whole country of the twentieth power of the population, 1905 to 1920. At this unbelievable rate, there would be 100 000 000 motors by 1930, a manifest absurdity. By applying reasonable adjustments, however, 30 000 000 motor vehicles can be foreseen, possibly by 1940, and, in the near future, present factory capacity will be needed to meet renewals and to supply the 5 000 000 new families by 1940, or 31 000 000 people in all. At present, there is in this country one motor vehicle for each telephone,—three motors to each five families—and in California one motor vehicle for each family.

If the California rate is reached for the entire country, the density just predicted will materialize. That this is not improbable is indicated by the fact that the average motor density in cities and towns above 1 000 is already 1.6 families per motor and about 1.7 in those above 2 500. This latter "urban" census group of cities contains about 60% of the nation's automobile registration.*

Meanwhile, cities are experimenting with block signals, one-way streets, etc., usually without adequate facts, surveys, or design, and, in desperation, the police adopt "no parking" as the last resource. Detroit, Mich., the motor city (since operating its own municipal railway system), has laid this prohibition on all down-town streets and all the twelve main radial streets for several miles outward, 40 miles of streets in all, that is, the hub and spokes of the business wheel.

* Report of National Automobile Chamber of Commerce, based on the "Motor List."

This is not engineering; it is a stampede. Where are the sober facts, surveys, and carefully worked out plans of experienced authority that justify such drastic measures from a technical or economic viewpoint? The cost of planning would be microscopic, relative to the saving.

How Shall the People Be Served?—Obviously, density regulations will fail without transportation service. Private motors will be used until central congestion stagnates everything. It is evident that the needs are not being met, as every new street and transit facility is so quickly overloaded (for example, Michigan Boulevard in Chicago, and the new subway in Brooklyn, N. Y.). In fact, rapid transit concentrates traffic, increases land values, and makes more congestion, unless it is planned and "integrated" with all other facilities and methods. This broader planning of transportation seems to be the only key.

The Terminal System Controls Capacity and Routing.—The "bottle neck" seems to be the stumbling-block. Railroad men know that more line capacity without terminal capacity is virtually useless. Traffic and transit follow the same dynamic laws—an open book to those experienced in their application. Chicago has only a 2-track loop to serve as a terminal for a dozen or more elevated tracks converging thereon. As a last resort, through-routing was adopted—a mechanical necessity. At present, the station-loading time is the final limit of capacity. The same is true with street cars and bus lines, except for the added obstruction of the police signal time. Street capacity is a scientific problem of the same nature. Cities are still operating 2-track traffic, like the Chicago Loop.

Thirty-Minute Time-Zone Controls Major Settlement.—In transit, 5 min. saved expands this zone 1 mile and 10 min., 2 miles, thus doubling, automatically, the heavily settled areas in most cities. Zoning can control the expansion, but without adequate transport service, zoning must be relatively ineffective. In New York this was recognized in the first zone planning. People turn to rapid transit and railroads when the streets become too slow. Thus, the 30-min. time-zone extends 15 miles along the Chicago North Shore by train, but only 4 or 5 miles on the surface. This is the economic control which no amount of good zoning can, by itself, overcome.

Functional Purpose of Zoning in Transportation.—In order to distribute population rationally, reduce central congestion, relieve trunk thoroughfares of traffic "nodules" or bunched congestion, make possible adequate transportation, speed up that service, and reduce economic waste. What is the measuring stick of "density"? For on the density rests the transport capacity needed. If the limits are too easy, transport lines are overloaded, streets cannot carry their burden, transit is stagnated. It is no solution to invade other streets of the city with many new lines of traffic of equal inefficiency, even if such new lines could be financed. Obviously, to obtain adequate street and transit capacity, higher and higher zoning limitations are required. The widening of a few streets or a few blocks in the central district will not suffice for immediately the building height limit shoots skyward several times

faster. The outside zoning density standards in the end form the major control.

Land Values Measure Character and Use.—The entire trend is toward congestion. Which is cause and which effect is now immaterial. Business has encouraged congestion and is now dispersing in desperation. Fig. 6 tells the story. This city is not yet developed to one-third its full building height under the zoning ordinance. What will happen when it is? It has a "twilight zone", or blighted district. The next parallel streets are as fully suited for high-class buildings as "Main Street". Merchants demand car-line terminals in front of their stores, private "no parking" signs, practice curb loading of merchandise, and look with "envy, hatred, and malice" on the struggling commerce of competing streets. Where is the "ceiling"? Why not fill up the valleys and develop the slopes?

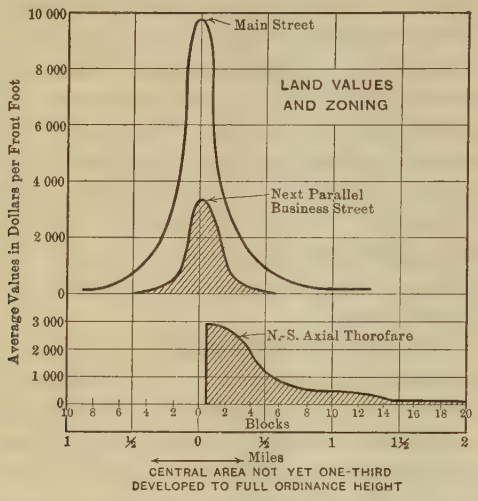
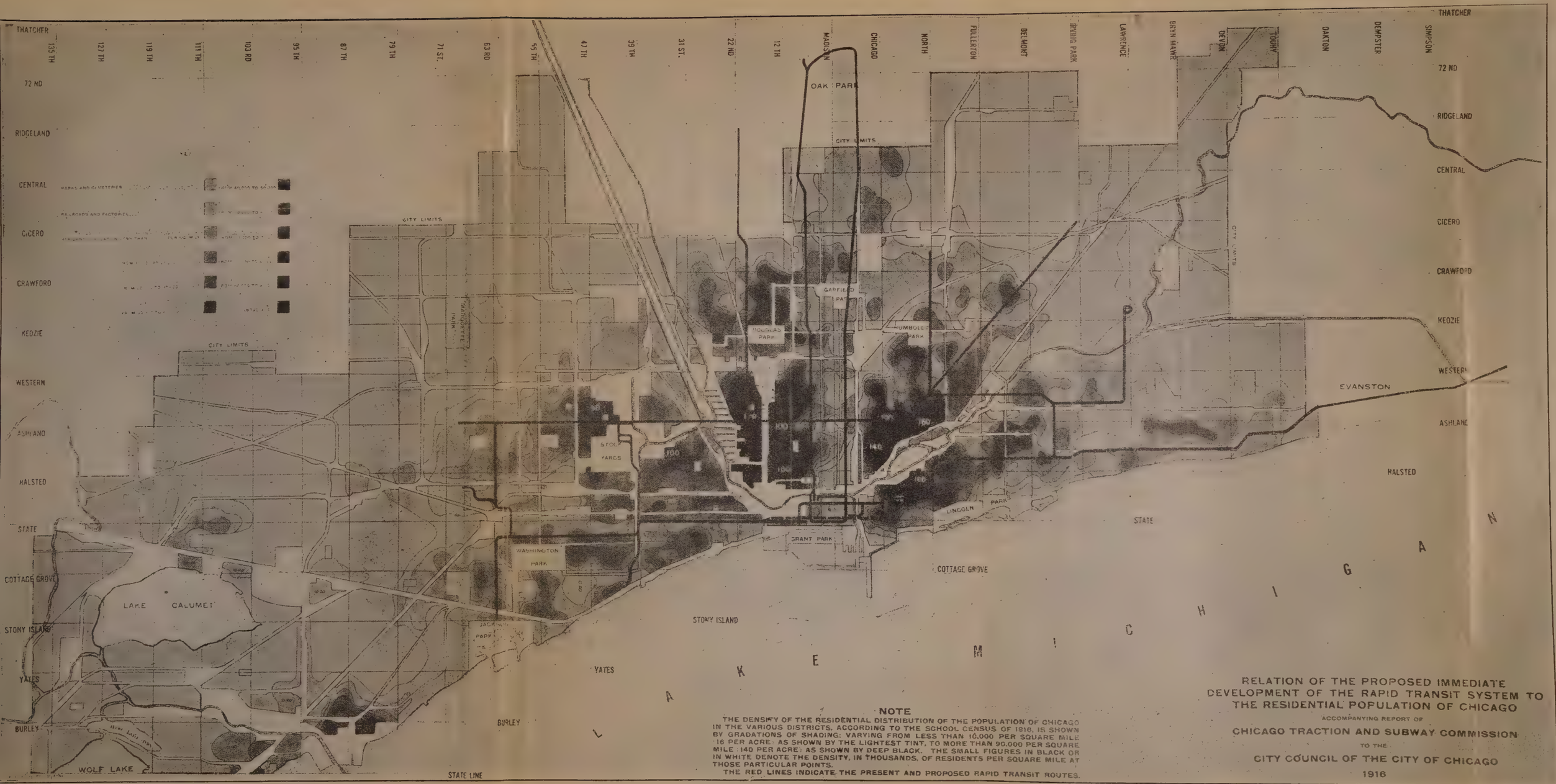


FIG. 6.

Density Profiles Give Definite Characteristics.—Fig. 7 shows clearly the basis which should underlie both zoning and transportation service. It applies equally to the large and the small city. The opportunity for control in the small city, however, is unlimited. In the large city, it is very limited because of established precedent. The City of Chicago is 26 miles in diameter, excluding the suburbs. (Manhattan, superimposed, would only reach half way, that is, from the Stock Yards to the northern city limits.) The population always concentrates nearest its work. The transit fare and the trip time control this concentration more than anything else. Relieve transit, and the time-zone (for the same fare and operating cost) is expanded. Zoning forces expansion without increasing traffic capacity at the throats where it is most needed. It cannot decrease present congestion without such relief. It is not retro-active.

Fig. 8 shows progressive density profiles for Chicago, and Plate II is a topographic density map of Chicago in relation to rapid transit development.

ZONING AND TRANSPORTATION SERVICES.



NOTE

THE DENSITY OF THE RESIDENTIAL DISTRIBUTION OF THE POPULATION OF CHICAGO IN THE VARIOUS DISTRICTS, ACCORDING TO THE SCHOOL CENSUS OF 1916, IS SHOWN BY GRADATIONS OF SHADING: VARYING FROM LESS THAN 10,000 PER SQUARE MILE (16 PER ACRE) AS SHOWN BY THE LIGHTEST TINT, TO MORE THAN 90,000 PER SQUARE MILE (140 PER ACRE) AS SHOWN BY DEEP BLACK. THE SMALL FIGURES IN BLACK OR IN WHITE DENOTE THE DENSITY, IN THOUSANDS, OF RESIDENTS PER SQUARE MILE AT THOSE PARTICULAR POINTS.

THE RED LINES INDICATE THE PRESENT AND PROPOSED RAPID TRANSIT ROUTES.

RELATION OF THE PROPOSED IMMEDIATE DEVELOPMENT OF THE RAPID TRANSIT SYSTEM TO THE RESIDENTIAL POPULATION OF CHICAGO

ACCOMPANYING REPORT OF

CHICAGO TRACTION AND SUBWAY COMMISSION

TO THE

CITY COUNCIL OF THE CITY OF CHICAGO

1916

Since 1860, the population crests moved outward from the 1 to 2-mile radius to the 4 to 5-mile radius in 1916, and to the 5 to 6-mile radius in 1920.* For perhaps twenty years, there has been no increase in the trunk-line transit capacity of streets, except of a minor nature, but congestion and average haul have constantly increased. Chicago still does business on the mountain peak of land values—the Loop, 8 by 12 blocks—with an atrocious blighted district surrounding it. A significant change, however, is coming, for it was recently reported that merchant advertising outside the Loop amounted to one-third of that of the Loop stores. Shades of "State Street"!

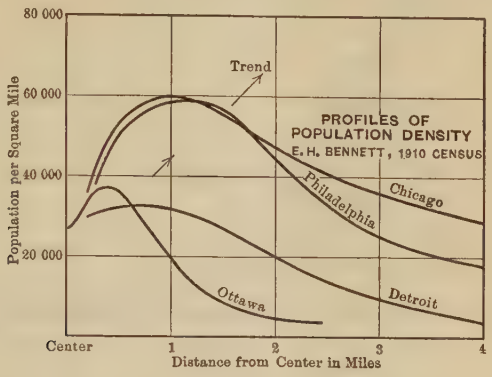


FIG. 7.

What Can Zoning Accomplish?—Zoning seeks control through use, height, and area (or volume) limits, an excellent basis, especially the set-backs, both in residential and high building districts. This height is usually a multiple of the street width and automatically increases if the street is widened. Widening, however, usually has little effect on track capacity (except in relieving traffic interference), because the street is still limited to double-track traffic. The central district densities in Table 3 serve to illustrate ultimate possibilities.

The possible densities shown in Table 3, or even half of them are appalling. The half-mile zone in Chicago, the Loop, is as yet only half built up; and in Indianapolis, it is only about one-third the full ordinance height. Ten-car rapid transit trains, of 125 people per car, with 1-min. headway, have capacities of only 75 000 people per hour; with 30-sec. headway, 150 000 people per hour. For such densities, surface transit is hopeless and bus transit still more so.

D. L. Turner, M. Am. Soc. C. E., reports that the New York steam railroads handle daily a passenger transit traffic about equal to 20% of the entire railroad traffic of the United States. The late A. H. Smith, President of the New York Central Railroad, predicted a catastrophe in New York within the decade, if the passenger transit problem were not adequately dealt with. Height zoning seems one of the most effective measures if stringent enough, and progressive height taxation has been proposed as an additional curb. The

* "Extension of Chicago Traction and Subway Commission's Curves", by Mr. R. F. Kelker.

skyscraper is already declared to be an economic misfit. It has been stated by F. H. Bartlett, President of the Chicago Real Estate Board, that:

"The tall building is responsible for raising ground value above the possibility of the earning power. * * * The skyscraper in Chicago is an economic financial blunder."

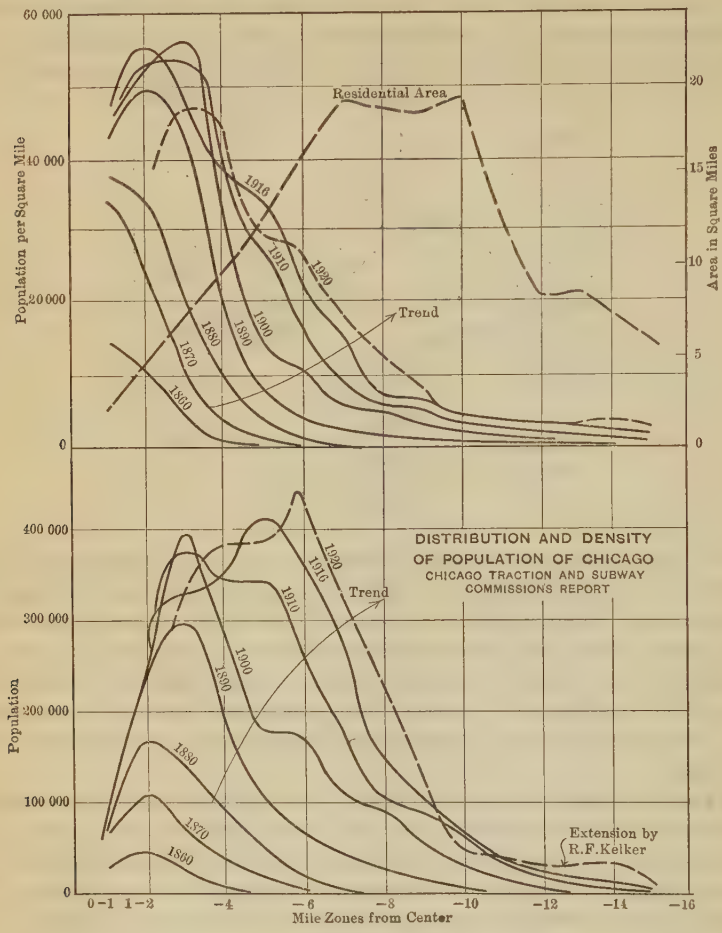


FIG. 8.

Area-density zoning for the residential districts is an excellent control if the limits are placed sufficiently low. The density limits of the outer zones seem to range from 3 750 to 5 000 and 7 500 sq. ft. per family. For the close-in or "peak" districts, the limits range from 300 to 600 sq. ft. per family, or are "unrestricted". The results are illustrated in Table 4.

Translate Table 4 into results. Fig. 9 shows (1) the possible or ultimate density; and (2) the actual density to-day for a city of about the 2 000 000 class. The densities are gross, streets included. The upper curve shows the zoning ordinance standard, and the lower curve, the profile of a 1-mile strip

along the most densely settled thoroughfare of the city, thus giving tributary population to the transit line bisecting it. Thus, a "5 000" zone permits a gross density of about 14 000 persons per square mile; a "2 500" zone permits 27 000; a "1 250" zone 54 000; and the "apartment" zone more than 100 000 per square mile. The present densities are far less than these limits, being 21 000 maximum in the second-mile apartment district.

TABLE 3.—Building Heights Allowed.

Street width, in feet.	Height Times Street Width, in Feet.				
	1.	2.	3.	4.	
60	60	120	180	240	Buildings with set-backs could exceed these heights.
80	80	160	240	320	
100	100	200	240	400	
	Occupants per Square Mile, in Millions.				
60	1.1	2.1	3.2	4.4	Twelve blocks per mile; 80-ft. streets —35.6%; 9 900 000 sq. ft. net; 100 sq. ft. per person; 10 ft. per story; 278 000 people per story.
80	1.4	2.8	4.4	5.7	
160	1.8	3.6	5.3	7.1	

This means that these limits permit a population from two to eight times or more in present areas. Zoning does not yet operate, by itself, to "spill over" excess population into adjacent zones. This city regards itself now as "congested". Its maximum square mile density is 21 000. The A_5 District permits well over 100 000. In 1916, Chicago's Ghetto had a density of 160 000; that of New York, 640 000.

TABLE 4.

Density limits, in square feet per family.	Size of Lot Permitted, in Feet.					Population per Square Mile.	
	One family.	Two families.	Three families.	Four families.	Five families.	Net.*	Gross.
7 500	50 by 150	.....	.....	.....	.....	15 000	9 000
5 000	40 by 125	.....	.....	.....	.....	23 000	14 000
3 750	.....	50 by 150	.....	.....	.....	30 000	18 000
2 500	.....	50 by 100	50 by 150	.....	.....	45 000	27 000
1 250	.....	.....	40 by 94	40 by 125	.....	90 000	54 000
625	.....	.....	.....	30 by 83	35 by 90	180 000	108 000

* Assuming 40% street and park area deducted from gross area.

Thus, the limits in the inner zones are the most important from a transportation outlook. They control or congest the throats of traffic. The maximum limits on the outer zone are of least importance, because only a small number of people are affected and these are inclined to use private means of transport.

Transit service to this 1-mile strip containing 70 000 people would require about 30% more track capacity than a 2-track surface line could effectively

render. Actually, two car lines serve it in addition to an automobile service that is actually carrying a rush-hour traffic as great as the transit system, taking the business district as a whole. What will happen with two to five, or more, times the present density?

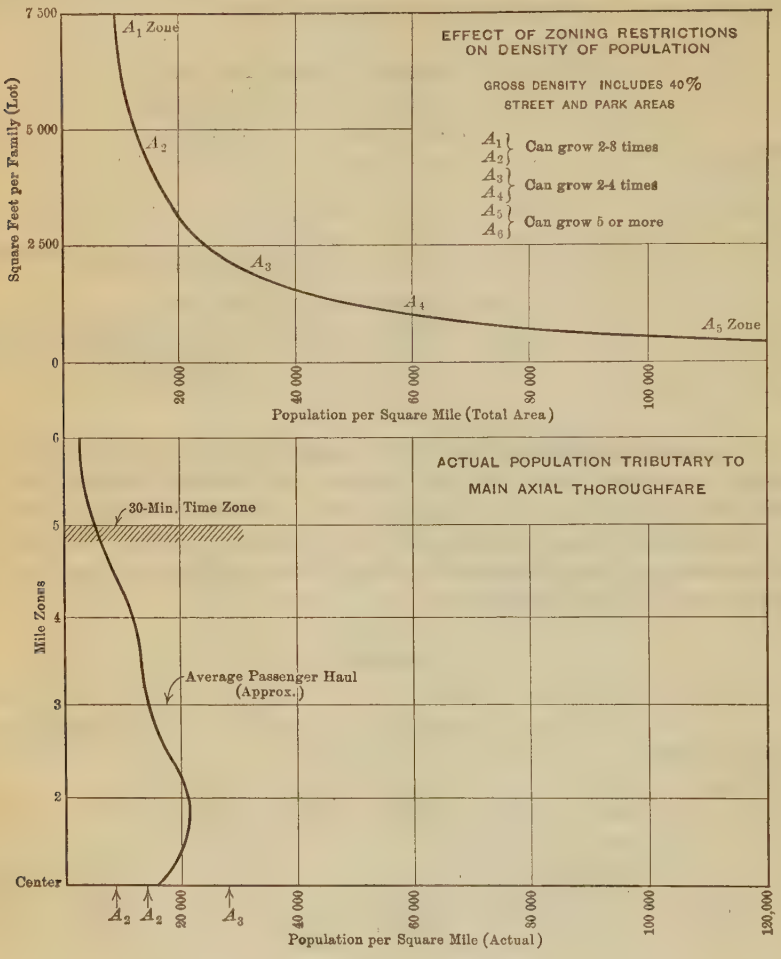


FIG. 9.

Zoning by Transportation.—Fortunately, there are some quick methods of control, known to transportation men. Let us pass over the zone-fare system at once, for it is centripetal, not centrifugal in its action, like a rubber band, urging people toward the center to reduce the cost of transport. A higher fare for the whole city brings a more wholesome development. Apparently, Americans will not tolerate European practice in this regard.

The zone system of short-lines, or tapered service, however, can exercise control if the service is extended somewhat beyond the present needs and

short terminals are located so as to encourage the building up of proper sub-centers. This system is economical, and conduces to transport efficiency—maximum service for the lowest fare where needed. It is always opposed by real estate speculators just beyond the sub-terminals. Down-town short-line terminals have the same advantage and tend to expand the central business districts.

Zone control can also be accomplished through special co-ordinated transport services—cars in the denser areas, bus feeders beyond, and limited-stop express busses (with differential fares) for long haul to established sub-centers, as in rapid transit express service. These methods make the best use of the limited street capacity of the throats as well as the track capacity. Such a zone-fare system is justifiable.

Recapitulation.—What zoners might consider:

1.—The establishment of an economic "ceiling" in the central districts that will padlock the "mountain tops" more effectively. There is plenty of "camping ground" on the slopes and in the valleys.

2.—More drastic area restrictions in the close-in residential districts, because the present limits are of relatively little effect in dispersing the great mass of the people from the throats.

3.—Zoning might aim at positive control within double the present population, and not wait for it to multiply five or ten times. The lower the density to-day, the greater the opportunity of accomplishment.

4.—Zoning should be based on, and not precede, a thorough street, traffic, and transit plan, developed for a city at least double the present size, and with street and track capacities properly designed for the greater densities to be permitted by zoning—if they are permitted.

5.—Mass-decentralization will remain potential rather than real unless the "ceiling" is lowered faster or traffic deliberately permitted to strangle the central district and effect its own crude cure.

What the Transportation Plan Can and Should Do.—

1.—Exercise more positive control over excessive density and throat concentration to serve best and develop various tributary regions of the entire metropolitan area.

2.—Establish operating methods and services best suited to the needs of various classes of patrons (including equipment, service schedules, routing, speeds, loadings, stops, fares, transfers, train operation, etc.)

3.—Design terminal routing to obtain more uniform density of occupancy in the central district, facilitate easy movement across it, and encourage logical outlying terminal sub-centers, cross-town lines, etc.

4.—Plan transport services as far in advance of proportionate settlement as may be warranted by the earning capacity permitted for the older established part of the system. In view of the restrictions and competition recognized by the public, it may be necessary to resort to partial local-district assessment for these outlying utility services, in proportion to the resulting benefit; or to relieve transit of the track-paving burden which often costs as much as the track structure itself.

Conclusion.—Zoning is vastly beneficial to orderly development, checks glaring misuses, and, as now administered, will ultimately exercise a restraining influence on mass density, but unless standards are raised, it cannot hope to turn the tide of growing congestion promptly enough to meet obvious traffic needs of the immediate future. Zoning needs transportation to effect real decentralization.

Transportation, if designed for the purpose and intelligently administered, offers the quickest and most positive control, including thoroughfares, traffic organization, transit, motor-bus, rapid transit, interurban, and railroad commuter service and terminals. All are part of the control system and should be developed together, not in competition.

This metropolitan control is so critical, in present and future aspects, especially "traffic congestion"—the clear result of unpreparedness—as to warrant placing this important phase in the hands of a skilled technical commission, adequately financed and allied with all city plan developments, having responsible charge of the design or planning of such an integrated transport system as will provide most promptly this desirable control and schedule the most practicable future steps. Such a commission should be sympathetic to and support the zoning and city planning movements for the more rapid progress of all. Without adequate transport facilities, all will fail of more than potential achievement.

TABLE 5.—Zoning Re-Allocation in a Large City.

	Percentage before zoning.	Percentage after zoning.
One-family houses	12	3
Two-family dwellings	8	19
Apartments	4	13
Commercial	5	14
Manufacture	13	25
Streets and alleys	23	23
Parks	3	3
Vacant and other	31	none

Summary: Dwelling area: Before zoning, 24% of total city area.
After " 35% " " " "
Business area: Before " 18% " " " "
After " 38% " " " "
Streets and parks: Before " 27% " " " "
After " .... no increase in area.
Net usable area for residence or business.....73%

Increase from zoning: Apartments.....186% area.
Two-family.....146% "
Commercial.....184% "
Manufacture..... 88% "
One-family..... 76% (decrease)

Approximate density: Population per square mile before zoning:

One-family	24 200	gross area		31 500	net, less street	
Two-family	49 100	"	"	63 500	"	"
Apartments	83 500	"	"	106 000	"	"
Average	43 500	"	"	56 500	"	"

Approximate distribution of population before zoning:
One-family.....27%
Two-family.....36%
Apartments.....36%

It must be apparent that rail and industrial traffic—passenger, freight, steam, and electric—is rapidly shifting to public streets and highways. The element of proper economic compensation for use can be determined on an equitable basis. The point is, this new traffic burden is here or is on the way. Cities average 800 miles of paved streets per million people, some having 1 500 to 2 000 miles.* This means a cost of $40 000 000 to $100 000 000 per million people served. City population will increase 25 000 000 by about 1940. "Traffic" multiplies many times faster. Increased transport capacity must come through (*a*) more "right of way"; or (*b*) higher use-efficiency. The cost of the former method is staggering. Which shall it be?

Table 5 shows the re-allocation of areas in a large city recently zoned. Note that with 143% increase in residential, 186% in apartment, and 186% in commercial areas, street areas remained static.

American cities should take warning from the present débacle of London traffic, with its entire lack of co-ordination and control. This great public issue has reached a stage demanding the highest technical skill and broadest experience, just as in zoning and other specialized phases of the metropolitan plan.

* The "Municipal Index", based on survey by The Asphalt Association.

ZONING AND WATER SUPPLY

By H. Malcolm Pirnie,* M. Am. Soc. C. E.

Zoning in relation to water supply is not new, and water supply engineers are familiar with the idea. Growing cities in hilly country early found it necessary to divide their distribution systems into low and high-service districts, and successive building developments occupied the areas adequately served with water. In these cases the water supply exerted considerable influence in restricting periodical growth within certain limits of elevation. The shores of many rivers, due to the need of large quantities of water for manufacturing purposes and the development of available water powers, became factory zones in the infancy of cities. In fact, the location of many cities was primarily due to an adequate water supply for industrial purposes. The height, bulk, and inflammable nature of buildings determine the water facilities for adequate fire protection and where these have been regulated by building codes, fire protection zones exist, in which different water pressures, capacities of mains, and types and spacing of hydrants are provided. Since the general acceptance of the germ theory of disease there has existed also an exercise of the police power over the use of private property lying within the watershed areas of public water supplies to prevent their pollution, which established zones of sanitation.

The question under discussion has to do with the relation between zoning for height, bulk, and the use of buildings and the water supply. Obviously, the well ordered zoning of growing communities will effect considerable saving in first cost and cost of reinforcement of the distribution systems. If the plan of a city indicates the limits of its industrial sections, its areas dedicated to high buildings, to apartment houses, to closely built two and three-family houses and to one-family houses, and the zoning ordinance places limits within the various areas on the height and bulk of the buildings, the present and future water requirements of the areas can be closely estimated. The distribution system may then be designed for the ultimate development and parts of it installed from time to time as required. If the early development of an area is scattered, a few of the ultimate mains may be laid with small, short-lived, steel pipes in intermediate streets. In this way, every cast-iron main that is installed will remain in service indefinitely. Before streets are paved, the proper water main with service connections, gates, and hydrants, can be installed.

Streets must accommodate pedestrian and vehicular traffic, street-car tracks, elevated and subway rapid transit lines, electric light and power cables, telephone, telegraph, and signal conduits, sewers, gas pipes, and water mains, and, in some cities, the allocation of space above and below ground adequately to provide for these utilities, has become a serious problem. New York found it expedient to drive a pressure tunnel under Manhattan Island, with

* Cons. Engr. (Hazen & Whipple), New York, N. Y.

risers connecting with the existing distribution mains to supply the large and increasing consumption in Lower Manhattan and Brooklyn. In spite of the great cost of this construction, it was demonstrated to be much less than the cost of equivalent service through additional pipe lines, even if it were possible to get them into the already crowded streets. A similar tunnel has recently been recommended to reinforce the overtaxed distribution system of Philadelphia, Pa. Such difficult works are the result of disregard for the proper relation between the width of streets and the height, bulk, and use of buildings on the abutting property.

The past uncertainty as to the trend of building has resulted in the replacement of small mains with large mains under the most difficult construction conditions. The removal of pipes that could have served the original type of development for many years, the cutting and replacing of pavements, and the interference with traffic, are constant sources of economic loss in American cities. When a residence street that was adequately served with a 6-in. main gives way to a block of apartment houses and hotels, lots that were formerly supplied through ¾-in. service pipes call for much larger services. The use of flush-valves in these buildings increases the peak demand for water, and the resulting inadequacy of the water main sometimes is not discovered until after an expensive pavement has been laid. The effect extends to connecting mains and the remedy often requires the cutting of pavements and the replacement of small mains with larger ones throughout a considerable area. The construction of high apartment houses in a relatively high residential area may make it necessary to install a booster pumping station and establish a high-service district. Such an area formerly may have been adequately served with water under the general distribution pressure, but with the substitution of high buildings for low widely separated houses comes the need for greater pressure for fire fighting and for giving service on the top floors. Thus, among the many unnecessary costs attendant on haphazard city building, the wasteful expenditures for water facilities command an important place. Stability in existing water-works structures and in present provisions for the future water supply of a community is dependent on stability in building, and may be secured by the early adoption of a carefully prepared zoning plan and ordinance.

In preparing a zoning plan, the water supply must be given due weight with all other purposes in view. Water has been and still is about the cheapest of all public necessities. Its cost in the future, however, will be greatly increased in some communities if the city plans are made without consulting the water authorities. It is possible even to destroy an existing source of water supply by disregarding this phase of the problem, thus burdening the city with a cost for new works out of proportion to the benefits resulting from zoning.

A striking instance of what can happen to a city water supply when it is lost sight of in the midst of other, and for the moment more spectacular, developments exists in the present situation at West Palm Beach and Palm Beach, Fla. These rapidly growing cities are situated on the east coast of

Southern Florida, somewhat more than 40 miles east of Lake Okeechobee. On the western edge of West Palm Beach, about 2 miles inland from the Atlantic Coast, are two shallow, fresh-water lakes, which are splendid natural impounding reservoirs. West of the lakes are the Everglades, with a general slope of 1 ft. in 4 miles from Lake Okeechobee to the coast. The West Palm Beach lakes are in one of the natural outlet depressions which formerly discharged great quantities of water from the Everglades and were capable of supplying water to a population many times that of the present cities. The Everglades drainage operations have recently diverted the flow from the lakes and the area remaining naturally tributary to them is not large enough to furnish the water needed now. In order to avert a water shortage, it has been necessary to purchase large tracts of land and to construct several miles of canals by means of which some of the lost drainage area has been reclaimed.

If some method can be found whereby the needed catchment area can be protected from further deep drainage operations, the present source of supply can be saved. Otherwise, the cities must choose between a supply from Lake Okeechobee, pumped through a 45-mile pipe line, and a supply from the West Palm Beach Drainage Canal which is subject to considerable pollution, is extremely hard, and would require expensive treatment. It is clearly best to conserve the present supply and to reclaim additional drainage area by extending canals as required to meet the growing demand for water, provided the investment in such works is justified by zoning the area against other drainage operations.

Under Section 3 of the Standard State Zoning Enabling Act of the U. S. Department of Commerce, it is pointed out, among the other purposes in view, that the zoning regulations shall be made in accordance with a comprehensive plan and designed to facilitate the adequate provision of water. The adoption of this Act in Florida would make it possible for the City of West Palm Beach to establish a water supply zone by extending its limits to the west. The size of the zone would be such as to furnish water for a long period in the future. Within the zone the lowest lands would be set aside as dredging areas from which the material would be obtained to fill the intermediate land. The water areas thus formed would become lagoons of the lakes or canals, and marginal parkways would be established to protect the water from pollution. If houses were built in the area near the lakes their sewers could be extended out of the area to points north or south.

A comprehensive plan of development of such a water-supply zone could be effected that would benefit all lands within it and, at the same time, regain a supply of good soft water sufficient for many years. The marginal parkways would furnish several miles of attractive drives near the cities, and the bordering land with its view of parks and lakes would have great value as residence property. The fine deep sand underlying this area would largely purify any unavoidable pollution on it, and keep the water in the lakes in satisfactory condition for use as a water supply after filtration. The lagoons and canals would be of such capacity that the ground-water level would be maintained sufficiently near the surface to support vegetation even in dry

seasons. The elimination of the shallow swamps now covering the bulk of the area would destroy the mosquito-breeding grounds nearest the cities. There can be no doubt that such works would increase the value of all remaining property more than enough to compensate the owners for the parts of the area dedicated to lagoons, canals, and marginal parks.

In the adequate provision of water there are various degrees of aid that can be obtained through proper zoning. In the situation just outlined, the zoning of the catchment area is the only practical means by which an excellent and most economical source of water supply can be preserved. In all communities, however, there will be financial loss due to the lack of stability in water-works building, as long as haphazard city building is allowed to continue. The importance of zoning in stabilizing the water-works business and continuing water as a relatively cheap commodity, is just as great as it is in otherwise conserving property values and promoting the health, safety, morals, or the general welfare of communities.

DISCUSSION

MORRIS KNOWLES,* M. AM. SOC. C. E.—The speaker is struck with Mr. Brewer's statement that "any zoning ordinance should require every new subdivision to have at least a 2-acre playground for every 160 acres subdivided."

The question arises, is it wise to try to accomplish this through the police power of the zoning ordinance? It suggests at once the type of regulation so frequently attempted under zoning—an excellent idea in itself—but a problem which should come under the broader subject of city planning, leaving under zoning only the things that properly belong there.

The speaker has been much interested also in the discussion of the relation of zoning to telephone problems by Mr. Allen. The author holds that the rate-payer is the person who suffers. This fact is important in all public utility problems. It is the same with regard to the paving of streets by street railway companies; or the paying of toll charges for water lines or gas lines crossing bridges; or the pole taxes for which the electric light companies are assessed; or mileage taxes for gas companies having pipe lines in the streets. Some of the charges of this kind have aimed to provide municipal revenues, rather than to pay the cost of municipal inspection; from that point of view they are not economically sound. In attempting such assessments, the people have forgotten that it is the rate-payer who pays.

This is also true in the telephone business, illustrating an important consideration in zoning, namely, that in all attempts to be theoretically perfect, practical limitations must not be forgotten. For example, in a desire to zone thoroughly for residence districts, care should be taken not to impose such conditions as will make all the residents pay additionally for this common and ordinary convenience. It seems as if there is no good reason why the buildings of a telephone system, when needed in a residence district, should not be considered in somewhat the same classification as a church, a school, a hospital, or possibly a fire station. These are not residences, certainly; but they are buildings which naturally are placed in localities where people live, because they are a convenience and because it is an advantage to have them in residential neighborhoods. Therefore, it is worth while to make a definite statement in the zoning ordinance with respect to this subject. The public who pays the bills is interested.

Frequently, one is met with the suggestion—as perhaps appears partly in Mr. Allen's paper—that if there is adequate legislation, by the aid of a thorough enabling act, and an ordinance that is sufficiently comprehensive, based thereon, with a Board of Appeals, then there is available all the machinery by which proper justice can be given when the time comes. However, just a little reflection will indicate that this is hardly practicable; although the speaker will confess that at first this was his own point of view.

In most situations, a telephone company must scheme to secure sites, far in advance, for the future location of central stations. If in that case it announces what it wishes to do by appearance before a Board of Appeals to

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

secure permission, the holders of property in that immediate vicinity will advance the price, and it will become increasingly difficult to secure property at all, or to secure it at a reasonable figure. In the end, the public will pay, and hence it is apparent that some other means must be adopted.

For a long time the speaker was somewhat puzzled as to why there should be any distinction between a telephone system, and an electric light, a gas, or a water supply system. In the telephone system one pair of wires goes from the central station to each subscriber, that is, one wire goes and another comes back, while with a gas or water supply there is only one main along the street with taps for the house services. The complicity of the telephone arrangement naturally limits the area that may be economically controlled from any one central office; and unless the sub-stations can be placed at proper locations, as determined by design, the weight of copper multiplies rapidly and the cost becomes excessive, especially if the subscribers are a long distance away.

In Pennsylvania, some time ago, a committee was appointed to see what regulations appeared reasonable. This committee had an opportunity to listen to the explanation of the officers and engineers of the telephone company in Pennsylvania, and the criticisms and discussions of those who represesented the municipalities.

The report submitted was in part as follows:

"The Committee is convinced, after listening to and considering the able presentation of the problem involved in locating 'Telephone Central Offices' close to wire centers, that a real necessity may exist for such structures in 'Residence Districts' * * *.

"The Committee is of the opinion that 'Residence Districts' should be relatively small, and such that any part is not far removed from a 'Business District'. When such is the case, the need for 'Telephone Central Offices' in a 'Residence District' may be less acute than otherwise. In non-populous communities, where one central office will serve the entire telephone community for many years to come, the municipal authorities in their discretion may decide to omit a 'Telephone Central Office' from such list of permitted uses in 'Residence Districts', and under the head of 'Exceptions' such use may be granted under appropriate restrictions. In such cases the Board of Appeals should be given the power to make a variance in the case of a special application and allow a 'Telephone Central Office' in a 'Residence District' if, in its opinion, it will not be unreasonably injurious to the surrounding property.

"However, while the making of 'Zoning Maps' is progressing toward accuracy in definition, in view of the application of zoning as now practiced upon maps and to meet the border line cases between municipalities, the Committee is of the opinion that the clauses given under (2), (3) and (4) are proper to have inserted and so recommend."

Clause (2), to which reference is made, reads as follows:

"(2) That in the list of definitions, under the suitable section of a drafted zoning ordinance, there be inserted, in the proper alphabetical order, a definition of 'Telephone Central Office'.

"*Telephone Central Office.*—A building and its equipment, erected and used for the purpose of facilitating transmission and exchange of telephone messages between subscribers, and other business of the Telephone Company; but in a residential district not to include public business facilities, storage of materials, trucks or repair facilities, or housing of repair crews".

However, as was stated by Mr. Allen, possibly the Company should be permitted to erect buildings of that type, higher than the customary limit within the district, and that they should occupy a larger percentage of the lot. The general feeling of the Committee was that this is the type of regulation which should be controlled by the Board of Appeals, because the Company could first secure the land without publicity, and, therefore, at a reasonable price, but, after having secured it, and knowing that it was safe, it could go to the Board of Appeals and make request for any necessary changes in height or limitation of area. It would have to show good cause, and the Board of Appeals would have the right to impose certain conditions. Although an ordinance may not under most jurisdictions impose conditions which are unesthetic, or out of harmony with the neighborhood, the Board of Appeals may impose some conditions as to the character and details of construction, such as the percentage of window openings, the restriction that a door may not open on a certain street, or any provision that would overcome conditions that might be objectionable in a residence district. Such restrictions could not be imposed by the ordinance.

The report concludes with Clauses (3) and (4) previously mentioned:

"(3) That, where the municipal authorities deem it necessary, as above stated, there be listed under permissible uses in 'Residence Districts'—whether classified in one or more different districts—'Telephone Central Offices'.

"(4) That there be included under listed 'Powers of the Board of Appeals', preceded by the inclusive phrase, 'In appropriate cases the Board may cause to issue a permit'.

"In connection with the erection and use of a building in any Residence District for a Telephone Central Office, for such variation of height and area restrictions as may be necessary for the suitable use of the structure; * * * and not unreasonably injurious to the property of the district in which the structure is proposed to be located."

Such provisions in a zoning ordinance will permit all that is reasonably required. It will be possible to purchase the land in the desired locations without publicity and, therefore, at a reasonable cost; the Board of Appeals, upon application, may grant such extensions to height and area as may seem desirable; and in doing so may impose such conditions as may be needed for the protection of the surrounding properties.

Albert P. Allen,* Esq.—The transportation system in a city, and especially its system of fares, undoubtedly has, or can be made to have, a noticeable effect on the distribution of population. Incidentally, the proper distribution of population throughout the available residence districts surrounding an economic business district is one of the principal objects of zoning; and the tendency to-day in all large cities is to insist that the street car fare must be a flat or fixed fare, with universal transfers, allowing a passenger to ride from one side of the city to the other at no greater cost than is required to ride a few blocks. In the speaker's opinion this system is absolutely wrong from an economic standpoint, and also from the standpoint of the best interests of

* Commercial Engr., Illinois Bell Telephone Co., Chicago, Ill.

the public itself, as indicated by the results that it has on the growth and distribution of population and the congestion of traffic on street and transportation lines which follows such distribution.

If street car fares were put on a more economic basis, which really means that the rate of fare should recognize to some extent the mileage involved, the change in the usual practice will be a great factor in establishing the proper distribution of population around secondary business centers, in keeping down the transportation between such centers, and in confining it within such neighborhood areas through the influence of the proposed mileage differential in fares.

For instance, in Chicago, a housewife living near the city boundaries can go down to the Loop District to shop with no more cost for transportation than if she went half a mile to a neighborhood store. This fact tends to congest transportation lines to the Loop and creates the greatest confusion there. If, however, she could ride to a neighborhood center for 5 cents and have to pay 7 to 10 cents to go to the Loop, there would be, no doubt, a noticeable reduction in the crowding.

The speaker remembers one case where a 2-cent charge for a telephone message has cut in two the number of messages previously handled on a "free service" basis. It might not seem that anybody would hesitate or change his practice for a differential of 2 cents, but unquestionably he will. Some rate differential in connection with local transportation is desirable, not only for the reasons already discussed, but also because it allows the transportation system to retain the cheap, but profitable, short-haul business which to a large extent is entirely lost when the average fare has to become 7 or 8 cents instead of 5 cents; and this fact in itself makes necessary an increase in the average fare which is greater than it would be if such short-haul traffic could be retained. It is quite probable, therefore, that the placing of street car fares on a logical, economic basis by paying at least some attention to mileage, possibly through a reasonable charge for transfers, will help the zoning of large cities through an improved distribution of population.

E. P. Goodrich,* M. Am. Soc. C. E.—Practically every paper presented as a part of this Symposium brings out by inference the definite need of establishing a relationship between zoning and transportation.

As a previous discussion (page 673) concerned the relationship between bulk and street traffic, so the same type of investigation must be made with reference to building bulk and the main transportation system, the street railway system, the busses, etc. Many of the items discussed by Mr. Bibbins will be of interest in this regard, but so far they have not been properly correlated. That is the speaker's only criticism of his paper; it is regretted he has not gone further to produce something more constructive.

In making that kind of a study, there are two or three methods which possibly should be followed. In Mr. Phillips' paper, the need of analyzing the zones of origin and destination, and of relating them, was suggested. That must also be done with reference to pedestrian travel, to vehicular travel,

* Cons. Engr., New York, N. Y.

and to rapid transit transportation. As yet there is no large public consensus of opinion, or system of facts, with reference to these things. The speaker gladly gives, for whatever they are worth, the results of certain investigations he has made, in the hope that other engineers will make similar investigations and publish the results.

Normally, the pedestrian walks as short a distance as he can, except during the noon hour. Then, there is a sauntering, recreational condition. Now, it is hardly to be expected that sidewalks will ever be designed for recreational purposes (except possibly at places like Atlantic City, N. J.); some other place should be provided (on the tops of buildings, perhaps), the sidewalk space, the street area, being set aside for movement for business purposes. The average distance which a person walks during other than the noon hour is about three blocks, and the rate is about 2.8 miles per hour. He goes more rapidly to his office in the morning than home at night. The floor area occupied by the average office is about 100 sq. ft. per person, including space for transients as well as for the permanent residents. One large building in New York, which may be considered as typical, because it has been checked in comparison with buildings in quite a number of other cities, discharges 1 700 people per hour. It occupies an entire block, and is 1 400 ft. in circumference.

Turning now to automobiles: One family in an average general residential district (including apartment houses) has the equivalent of about one vehicle per day coming to it and departing. This condition has been found to exist in all parts of the country and over large areas of varying economic character, it is an average figure. Obviously, in the districts of lower economic levels, the rate is less, but, in some other districts, it is correspondingly higher. Rural stores, or outlying stores, one story high, create and receive the equivalent of 1 vehicle per 5 running feet of frontage. The relationship which exists between the residential area and the store area is evolved as follows: On the average, one family is found per lot, 40 by 100 ft.; in outlying districts, the lots are larger. Toward the center, the apartment houses reduce the average area of land per family, but the figure given applies very generally over large areas of most cities. A store with a 25-ft. front will normally serve about 100 persons. The density of use of industrial areas varies considerably, but about 4 000 sq. ft. per vehicle per day has proved a fair average.

Now, when the proper zoning ordinance has been prepared for a community, so that the relative areas assigned to residence, business, and industry, are approximately balanced, so that there is an excessive industrial area with no available residences, or so that, as in some instances actually happens, a whole section of the community has been set aside for business and never can be used fully for business purposes, with a depreciating effect on the vacant properties and a most depressing effect on the business itself—when this proper relation has been deduced, then one can build up a street system, a drainage system, a telephone system, a recreational system, and all the other things discussed, to correspond with these particular types of development.

To-day most zoning requirements are too loose. They do not begin to make restrictions to the point which the evidence demonstrates that the needs

already demand. Generally speaking, building heights are excessive. In a few communities they are reasonable in comparison with street widths, but these cases are rare; most zoning ordinances are useless as far as setting limits within which one can design a street system and other facilities. The actual distribution must also be taken into consideration. There is only one ordinance among the one hundred with which the speaker is familiar under which there would be a limiting of the actual total population. The remainder are just starts in the right direction. To do anything with zoning that will affect the future with respect to transportation, for example, requires an entire change of attitude.

Respecting the problem of transportation and building height, assume a central distributing point, for instance for coal for the community, or a central market distributing food products, or a central district to which crowds come every day, as an office building district. If there is a low height limit—and, assume for this purpose, that the height is uniformly built up throughout the community—it is plain that the area required will be much larger than if there is a higher limit. Now, to supply coal from the central point requires an average haul which is considerably longer in the case of the low limit than in the case of the high limit; also, a longer telephone cable system, and a much larger water supply system. As to the streets, they will receive much more traffic, so that around the center a greater congestion will exist with the lower than with the higher height limit.

Obviously, judged solely on this argument, it is advisable to make all buildings just as high and just as compact as possible. That line of reasoning results in an extreme condition against which zoning is largely aimed. Somewhere there lies an economic balance toward which engineers should strive. The telephone companies have done more work than any other group on this problem. For example, the cost of displacing a telephone central from its natural economic location has been estimated for one specific case in an Eastern city. A diagram was prepared by the telephone engineers, giving the actual increase in capital involved by moving one, two, three, or four blocks from the natural center. If the telephone central station (serving 7 500 subscribers in this particular instance) were displaced ½ mile, it would involve a capital charge, which, at 6% interest, spread over the subscribers, would result in an increase of rates of 76½ cents per month. If there were a displacement of ¼ mile, there would be an increased charge of 26.6 cents per month which would normally be applied to the rates. Therefore, the question for the subscribers and for any community to determine is whether the benefits which will accrue from zoning are greater or less to the individuals as a whole than the extra costs incurred.

The same is true of street traffic. Is the congestion (which is costly) greater or less in cost than the remedy? The old adage that "the cure is often worse than the disease," often applies economically.

The paper by Mr. Brewer is particularly interesting from an engineering standpoint by furnishing data with reference to the use of playgrounds by individuals of different ages.

The subject of zoning is one on which engineers can well afford to spend much more time than they have in the past. It is one which needs the engineering method of approach, especially that of the economic engineer. The engineer is not merely a man who can design, but a man who designs for the least cost. The whole structure of modern urban life is so bound up with the economic features, with the cost (and zoning is the one thing to-day which makes an appeal fundamentally on an economic basis, although it is based also on health and safety and similar things), that the engineer from his economic point of view can well afford to delve deeply into it.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—More than two thousand years ago, using the deductive method of research, Euclid exhausted the subject of Geometry and Aristotle, the subject of Logic. Much of the science of City Planning also can be reasoned deductively; and much remains to be learned from experience. Deduction is useful: induction is absolutely necessary. Zoning is a new art the successful application of which depends on a careful engineering study of conditions as they exist. No two cities require the same treatment. Engineers are not yet equipped with codified principles and rules concerning the conformation of the districts of a zoned city, sufficient to govern procedure. Each zoning plan must be developed as a case by itself.

The subject under discussion is well stated by Mr. Brewer. Zoning influences each and all of the utilities mentioned, but each utility cannot be considered by itself in its influence on zoning; it must be studied in its relationship to each and all of the others. Thus, for example, the location of park reservations may be determined by that of storage and impounding reservoirs, the park protecting the water supply from contamination and the reservoir providing a scenic feature in the park. So, also, the street and transportation systems are intimately related; also boulevards and drainage.

The writer does not feel qualified to present the principles of zoning in its influence on recreation facilities in a systematic form. He will be content merely to venture a few statements from his own experience.

Connected parks imply a connected and continuous high-class or first residential district. The parks are connected by boulevards or parkways which provide valuable residential frontages. Zoning thus established is not subject in any considerable degree to controversy and, therefore, possesses great stability.

Connected parks also control to a lesser degree the second residential district which borders the first. This is due not only to the proximity of the first district, but also to the recreation facilities near-by.

A belt line, or surrounding boulevard or parkway, implies a belt of first and second residential districts. These districts are thus systematically located, which of course is a great advantage. In some measure the city thus acquires geometrical conformation. As boulevards and parkways frequently lie along watercourses, the improvement may be combined with that of storm-water

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

drainage. Brooks and creeks outside the city limits may be utilized in the location of a surrounding boulevard or parkway, the boulevard or parkway cutting across the intervening ridge from one watercourse which it has followed to another. Such watercourses are commonly edged with trees. The trees and water provide the architectural basis of development. These channels often are fouled by sewage and refuse, and, being useless for other purposes, can be secured at small cost. They may be dredged and regulated, and provided with drives and other architectural features. The removal of the nuisance is incidental to the improvement.

The artifice of building park "border mounds" is open to criticism in that they shut off park views from the surrounding streets and houses. The border mound properly functions in providing seclusion in the park by interposing a barrier of trees and shrubbery between the interior of the park and the ugly buildings surrounding it. Where the park is bounded by buildings of the first residential district the need of such a barrier is not so imperative. Seclusion can be had in sufficient measure by judicious planting, aided by a low mound set back from the street line. By proper architectural treatment a reasonable seclusion can be had for the park while at the same time permitting short views into the park from the outside.

Radial streets and boulevards reaching out to the surrounding boulevard or parkway imply expanding second residential districts developing outwardly into the first. Streamers from the business district will extend for some distance toward the surrounding boulevard or parkway. Between these streamers, near the heart of the city, will be found the light industrial district, and farther out the second residential district. Separation is natural and easy.

Promising areas can be encouraged to develop as they should by the establishment of parks and playgrounds, with boulevards leading centrally toward the commercial district and circumferentially from one park to another. The writer has in mind a most promising area on the west side of Houston, Tex., along the Buffalo Bayou, an area the value of which has not yet been fully appreciated. This might be converted into a first residential district with a parkway as a basis of development. The parkway should include the stream and be provided with drives on both sides. This part of the town is now one of the negro settlements; the natural asset of the stream is wholly lost.

Playgrounds, public gardens, and city squares are needed in the second residential districts between the radial streamers which extend outwardly from the business district. These are the gores where blighting is likely to spread with great rapidity. It can be checked in some degree by the establishment of recreation facilities.

Improper development of the heavy industrial district can be checked by judicious park layouts, saving the residential territory from serious blighting and keeping the heavy industries within due bounds near the railroads, where they properly belong. The railroads should be grouped and centralized, and not allowed to destroy the value of land better suited to other uses. At the same time, every possible encouragement should be given to the railroads and the heavy industries on which the prosperity of the city depends. The interests of

recreation facilities are not antagonistic to those of business and industry. There is no better method of aiding business and industry than by improving the living conditions of those on whom the success of business and industry depend.

C. E. Brewer,* Esq. (by letter.)—Mr. Knowles raises the question of whether the acquisition of property for recreational purposes should be a part of a zoning ordinance or left to the broader subject of city planning. The writer cannot understand why the acquisition of playgrounds is any less important to the welfare of a city than the regulation of the height of buildings or the proper lighting of an office building. In fact, for the future welfare of its citizens, the acquisition of playgrounds seems more important, because only a very small proportion of the population work in office buildings, whereas the entire population should have some form of recreation.

Mr. Knowles must realize that hundreds of cities adopted city planning programs years before the need of playgrounds and playfields was apparent or was recognized as being essential. The acquisition of playgrounds by cities that do not have them is now considered essential and must be done by legislation, that is, through condemnation. The writer thinks it would make a zoning ordinance more acceptable to the people and enhance the possibility of its approval by the voters if the acquisition of playgrounds were included; but the method of acquiring the property for recreational purposes is not essential; the main thing is to acquire it in some way.

Whether future recreation places are to be acquired as part of a city planning program or by a zoning ordinance, it precludes the adoption of a master metropolitan city planning program by the municipality. By a master metropolitan plan, the writer means that a survey of the country for 5 to 10 miles outside the city limits should be made and future recreation places determined as near an exact geographical distribution as possible and according to topographical conditions of the district. A careful study of the tendencies of the future growth of a city must be made. It would be impossible to determine long in advance whether a certain district 5 miles outside the present city limits would be residential, business, or industrial; but if a zoning ordinance that had adequate recreational requirements were in effect, the character of the outlying districts could be determined more readily in advance.

If a master metropolitan plan is adopted, the property designated for future recreational purposes could be acquired in a systematic manner, and would not be left to the whim and caprice of the realtor dedicating property to the city without such a plan. The property would also be acquired much more economically.

Two methods of acquiring future recreation sites are immediately presented, as follows:

1.—By education, that is, educating the subdivider to see that the dedication of a part of the subdivision for playground purposes is not only a civic duty and means much to the future growth of the community, but that it is also a profitable action to take, as lots and homes in a community having

* Commr. of Recreation, City of Detroit, Detroit, Mich.

recreation and school facilities, sell much more readily. There is also splendid advertisement involved in such a dedication.

2.—By legislation, that is, compelling by law each subdivider to set aside part of his subdivision for playground or park purposes. There are two methods which can be followed:

(*a*) By dedication outright to the municipality a certain part of each subdivision as required by law.

(*b*) The withholding from sale of a certain part of each subdivision as required by the master metropolitan chart. The land thus designated should be held in reserve and not placed on the market by the subdivider. The city should acquire this property when it needs it at the prevailing rate.

Method (*b*) is recommended as being the better because all future recreation places would be acquired systematically and economically. Method (*a*) would require each subdivider to give up a certain part of his subdivision, and it is doubtful whether this plan would be declared constitutional by the Courts; it might be construed as confiscation of private property. Furthermore, the least desirable land would be dedicated by the realtor and it might not be advantageously situated according to the density of population in that community. There would also be no correct geographical distribution.

All future recreational property should be acquired in accordance with the master metropolitan plan, and if reserved from sale by the subdivider, the city could acquire it when needed without condemning a large number of homes and expensive buildings. If it developed that this property was not needed by the municipality, it could be sold to private individuals without loss by the city.

The writer sincerely hopes that each city plan commission will make a survey of its metropolitan area and determine now its program for the future acquisition of recreational property. If this is done, a great deal of the confusion which Mr. Goodrich deplores, could be eliminated. The rapid increase of large industrial plants with 20 000 to 40 000 workers, with the transportation of large numbers to and from work at an acute stage, and the building of hundreds of large apartment houses, have brought to the city planner in every large city a huge problem of which he had no conception a decade ago. Zoning seems to be the only way out of the labyrinth of modern municipal living problems.

In his discussion, Professor Lyle further strengthens the argument that zoning should include the regulation of the acquisition of parks and playgrounds. He points out that parks and playgrounds when established, have some degree of stability. They are located in residential districts and when a family has acquired a home they leave it with great reluctance and only of necessity. Residential districts with adequate recreational and educational facilities do not fluctuate like business and industrial districts.

Professor Lyle's criticism of "border park mounds", that is, the planting of trees and shrubs sequestering parks and playgrounds, is well taken, yet there are three reasons for these border plantings:

First.—It is necessary from a "safety first" viewpoint, as they prevent children from running carelessly off the playgrounds and under the wheels of traffic, which often happens unless a fence is erected. Fences are often unsightly and do not fit in well with the usual beauty of park planting.

Second.—It serves as a screen from hideously painted houses, garages, garbage cans, etc.

Third.—It is a beautification and landscaping plan to develop the æsthetic sense of the people who come to the parks and playgrounds.

The only way to eliminate these conditions is to give to the recreation department or city plan commission, the power to regulate the construction and landscaping plans of all buildings and property within 300 ft. on all sides of the park or playground, as is done in Indianapolis, Ind. This plan is somewhat idealistic and could only be put into effect against the intense opposition of abutting property owners. It is again a zoning ordinance matter. The municipality must go as far as the property owner in the beautification of parks and playgrounds, which, in the writer's opinion, would be beneficial for all concerned.

Professor Lyle is again correct in his contention that recreational interests are not inimical to business, industrial, or residential interests. They are inseparable and make a complete cycle of welfare and happiness for the citizens, whether at home or at work, for there is no peace at home or in work, if the leisure time is not profitably spent.

The following information and statistics (Tables 6 to 12), which include the distances children travel to, and their attendance at, a playground is taken from a recreation survey made in Milwaukee, Wis., Minneapolis and St. Paul, Minn., Washington, D. C., and Detroit, Mich., by the Playground and Recreation Committee of the American Institute of Park Executives, of which the writer is Chairman. The results of this survey which was begun in the summer of 1924, are published herewith for the benefit of the members of the Society interested in this work.

Undoubtedly the great difference between the number of first visits and that of second visits as shown in Table 12, is due largely to the child not reporting present to the playleader or to the one in charge of the survey. Not counting the ones reported as visiting only once, the following tabulation gives an interesting percentage of the regularity of the children attending the playground:

15.7%	came	100%	of	the	time	
18.4%	"	90%	"	"	"	
22.4%	"	80%	"	"	"	
27.7%	"	70%	"	"	"	
32.8%	"	60%	"	"	"	
44.5%	"	50%	"	"	"	
55.2%	"	40%	"	"	"	
70%	"	30%	"	"	"	
99%	"	20%	"	"	"	

TABLE 6.—Distance Children Travel in Attending the Playgound, by Cities.

	¼ Mile.		½ Mile.		¾ Mile.		1 Mile.		More than 1 Mile.		Total.
	No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.	
Minneapolis, Minn.	469	34.1	474	34.5	220	16	98	7.1	113	8.3	1 374
St. Paul, Minn.	370	23.8	335	21.7	320	20.5	305	19.6	225	14.4	1 555
Detroit, Mich.	11 193	52.5	5 035	23.5	2 315	10.8	1 084	5.1	1 720	8.1	21 347
Milwaukee, Wis.	676	63.2	294	27.5	55	5.1	15	1.4	29	2.7	1 069
Washington, D. C.	2 996	33.8	2 300	26.1	866	9.6	963	11.0	1 724	19.4	8 849
Total	15 704	45.92	8 438	24.67	3 776	11.03	2 465	7.21	3 811	11.17	34 194

TABLE 7.—Distance Children Travel in Attending the Playground According to Ages.

Age.	No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.	No.	Percentage.	Total.
Less than 3 years	1 045	3.1	221	0.6	97	0.2	28	0.08	43	0.11	1 434
3 to 7 "	2 644	7.8	941	2.5	342	0.8	155	0.46	136	0.37	4 218
7 to 10 "	3 324	9.7	1 649	4.9	675	1.9	368	1.18	383	1.12	6 399
10 to 12 "	2 754	8.2	1 556	4.6	710	3.2	486	1.39	550	1.61	6 056
12 to 15 "	2 866	8.4	1 847	5.5	931	2.8	599	1.5	981	2.87	7 224
More than 15 years	3 074	8.7	2 221	6.5	1 021	3.0	829	2.5	1 718	5.09	8 863

TABLE 8.—Attendance at the Playgrounds, According to Group Ages.

Age.	No.	Percentage.
Less than 3 years	1 434	4.17
3 to 7 years	4 218	12.24
7 to 10 "	6 399	18.78
10 to 12 "	6 056	17.77
12 to 15 "	7 224	21.13
15 to 19 "	4 963	14.51
More than 19 years	3 900	11.40
Total	34 194	.....

TABLE 9.—Distance Children Travel and Percentage According to Age Groups.

Distance.	No.	Percentage.
Less Than 3 Years of Age.		
¼ mile	1 045	73
½ mile	221	16
¾ mile	97	6
1 mile	28	2
More than 1 mile	43	3
Total	1 434	..
From 3 to 7 Years of Age.		
¼ mile	2 644	63
½ mile	941	23
¾ mile	342	8
1 mile	155	4
More than 1 mile	136	2
Total	4 218	..
From 7 to 10 Years of Age.		
¼ mile	3 324	52
½ mile	1 649	26
¾ mile	675	10
1 mile	368	6
More than 1 mile	383	6
Total	6 399	..
From 10 to 12 Years of Age.		
¼ mile	2 754	45
½ mile	1 556	26
¾ mile	710	12
1 mile	486	8
More than 1 mile	550	9
Total	6 056	..
From 12 to 15 Years of Age.		
¼ mile	2 866	40
½ mile	1 847	24
¾ mile	931	13
1 mile	599	8
More than 1 mile	981	15
Total	7 224	..
More Than 15 Years of Age.		
¼ mile	3 074	35
½ mile	2 221	25
¾ mile	1 021	12
1 mile	829	9
More than 1 mile	1 718	19
Total	8 863	..

TABLE 10.—Distance Children Travel According to City Blocks.

No. of city blocks.	No. of children.	Percentage.
1	7 301	32.8
2	4 161	18.3
3	3 358	14.7
4	2 053	9.0
5	1 610	7.0
6	1 022	4.5
7	717	3.0
8	544	2.0
More than 8	1 992	8.7

TABLE 11.—Distance Children Travel to Playground According to Blocks and Age Groups.

City blocks.	Under 3 years.	From 3 to 7 years.	From 7 to 10 years.	From 10 to 12 years.	From 12 to 15 years.	From 15 to 19 years.	More than 19 years.	Total.
1	701	1 405	1 491	1 216	1 145	708	640	7 301
2	222	671	863	784	761	479	386	4 161
3	132	419	690	645	622	471	379	3 358
4	51	286	433	309	418	325	231	2 053
5	56	175	334	302	336	228	179	1 610
6	32	109	195	199	251	130	106	1 022
7	19	69	119	157	126	127	99	716
8	12	53	113	136	84	71	75	544
More than 8	42	99	294	280	361	340	576	1 992
Total	1 267	3 286	4 532	4 028	4 099	2 874	2 671	22 757

TABLE 12.—Number of Visits per Week of the Children Attending the Playground on a Ten-Period per Week Basis.

No. of times.	1	2	3	4	5	6	7	8	9	10 and more.
A City	3 175	752	191	197	150	66	39	30	21	48
B "	662	271	147	101	78	51	27	15	13	18
C "	234	217	133	91	79	43	72	41	25	225
D "	52	71	49	75	188	71	83	76	69	385

In marked contrast to this tabulation, is the careful study made on the Milwaukee Playgrounds. In a total number of 5 345 days there were:

908	days,	or	16.5%,	showing	no	visits	per day.
1 438	"	"	28%,	"	one	visit	" "
1 739	"	"	32%,	"	two	visits	" "
1 260	"	"	23.5%,	"	three	"	" "

In other words, Milwaukee had 23.5% of her children on the playground all the time; 55.5% two-thirds of the time; and 83.5% one-third of the time.

The survey showed that 63% of the children attending the playgrounds were males; and 37% were females.

J. Rowland Bibbins,* Esq. (by letter).—A careful reading of the entire discussion discloses that the writer's main objective has been realized, namely, to direct attention to and throw some light on the confused relation and practical possibilities of zoning in preventing growing congestion and in affording transportation relief, both surface and rail, and to show what zoning can and cannot do. A number of the papers and discussions made very frank admissions of these limitations (see appended excerpts). This frankness is refreshing and is the best earnest of future improvement. Imposing stringent zoning limitations is a hard task at best, as it runs counter to private interest and to individual profit at community expense—a form of public subsidy. It is clear that those engaged in zoning cities should have whole-hearted support in stiffening the regulations as to both height and density. They need help.

Although those who are working on different phases of the technique of city building should co-operate intimately, such phases should be recognized as distinct enterprises calling for the exercise of definite and specialized experience. This is particularly so in transportation, which has largely controlled, instead of being controlled by, broad movements of population and commerce. This condition cannot be changed over night, but it is the writer's belief, from study and long observation in most of the large cities of the United States and Canada, that more can be accomplished for quick relief and constructive city development by the technique of transportation than by any other method. Further, transportation can be made to build and rebuild defective cities along more rational and economic lines.

Mr. Goodrich observes with regret that, in the writer's presentation of these various forms of transportation controls, "so far they have not been properly correlated" and "that he has not gone further to produce something more constructive". Obviously, he fails to consider the limits of this Symposium, confined by title to the "Influence of Zoning on the Design of Transportation Services". Necessarily, the whole complex problem of design demands far more extended treatment than was possible, but under "Recapitulation" (page 715), a number of definite methods or controls of density and distribution were suggested, all of which are known and practiced by those experienced in the design, financing, and operation of transportation services by trolley, bus, rapid transit, or steam. No standardized remedies will fit all conditions. City transit is so vital to the community and conducted on

* Cons. Engr., Washington, D. C.

so small a margin of profit (essential to stable credit), that only well-matured plans, technically sound, can hope for adoption. For example, through routes, even if they are ideally suited to business decentralization and convenience, cannot always be adopted in full, however meritorious. Railway revenue is in no respect more sensitive than to proper routing, scheduling, and dispatching.

Again, although the matter of zone-fares or the mileage basis, which Mr. Allen apparently has in mind, is admittedly ideal, the writer is extremely dubious of either its practicability or fairness. However, the bus has developed an entirely new field of zoning through proper control of routes, stops, schedules, fares, transfers, terminals, loading points, speeds, and other details. To a degree this same result may be accomplished by trolley short lines, and by rapid transit, too, using specially designed surface highways with limited stops, as well as by express subways and elevated lines, all intelligently co-ordinated with the underlying surface system. This is a departure from the system of flat fares, for the benefit of those who require speed or special service and are willing to pay *pro rata*; but to establish a universal mileage-fare basis, under American practice, would breed intolerable congestion at the center of the city.

Messrs. Goodrich and Phillips refer to the apparent absence of "origin-destination" data as a prerequisite to adequate thoroughfares, transit plans, and other transportation features. It may be pertinent to state that this method has been used by the writer in many transportation surveys since 1910. In Rochester, N. Y., a "home-to-work" census of the business and larger industrial districts disclosed the city-wide residence of employees as a basis for the re-routing of transit lines and the allocation of service. In Chicago, Ill., 350 000 "Loop" workers were tallied, 500 000 elevated riders (in one day), and 1 500 000 riders on the surface lines, likewise as the basis of the design of the entire local transportation system of Chicago for both present and future development (Chicago Traction and Subway Commission). Similar surveys of motor traffic in Indianapolis, Ind., furnished basic data for a comprehensive "Thoroughfare Plan", including transit re-routing, bus routing, and a terminal system. In the development of a new railroad terminal and port plan for New Orleans, La., the total freight movement was tallied as to origin and destination, covering more than 50 000 carloads or about 2 000 000 tons of freight, foreign and local, to determine the proper design and location for various transportation services in their best relation to the city plan and thoroughfare system (Board of Advisory Engineers).*

This whole Symposium illustrates the existence of two types of approach to the technique of city building: First, the abstract principle; and, second, the concrete problem of design and application—the legal and technical; the qualitative and quantitative; possibility and probability; theory and practice.

* These matters have already been presented at length by the writer through various media including, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 693, *Official Proceedings*, New York Railroad Club, March, 1923; American Association of Port Authorities, December, 1923; *Engineering* (Philadelphia), May, 1924; *Engineering News-Record*; *American City*, and *Electric Railway Journal* at various times; and *Annals*, American Academy of Political and Social Science, November, 1924. They have also been reviewed comprehensively in a paper entitled "City Building and Transportation", presented before the Western Society of Engineers in 1920.

Transportation is organized as a complex, but unified system. It cannot be manhandled by piecemeal planning, except at the ultimate expense of the ratepayers. It lies at the very root of the entire city plan and until both architects and engineers give more recognition to this fact and properly integrate the transportation system—highways, bus, trolley, rapid transit, truck, railroad, terminal, port, industry—many city plans must prove abortive or suffer delay in their ultimate fruition.

The following excerpts from the Symposium are worthy of reiteration. On page 686, Mr. Phillips states:

"Zoning is based on conditions as they are, * * *. Zoning does not correct; its action is preventive. * * * Zoning in itself cannot be relied upon to take care of a bad traffic situation, but * * * may check the process of congestion * * *."

Mr. Leavitt has made the statement (page 668) that:

"Zoning a city before the city plan is devised may mean the eventual necessary abandonment of the best solution of the city plan, or it may lead to excessive costs in order to put into execution the best solution * * *. Zoning may bring disaster * * * by developing a peak load on inadequate streets, * * *. Zoning may best be done after, or as a part of, the city plan and not before. * * *."

Mr. Whitten states (pages 655-656) that:

"Bulk regulations cannot prevent this [crowding]. Crowded tenement conditions are likely to result. * * * Zoning regulations, supplemented by thoroughfare, transit, and transportation planning, can promote the decentralization of business and industry * * *. * * * The one thing that is clear is that congestion breeds congestion."

The following statement has been made by Mr. Crane (page 650):

"Of course, the design of the transportation system will have more to do with such decentralization than the limitation of building heights."

Mr. Goodrich states (pages 726-727) that:

"To-day most zoning requirements are too loose. * * * There is only one ordinance among the one hundred * * * under which there would be a limiting of the actual total population. * * * To do anything with zoning that will affect the future with respect to transportation * * * requires an entire change of attitude."

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1565

FOURTH PROGRESS REPORT OF THE SPECIAL COMMITTEE TO REPORT ON STRESSES IN RAILROAD TRACK*

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December 23, 1924

* Presented to the Annual Meeting, January 21, 1925.

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To the American Society of Civil Engineers:

The Special Committee to Report on Stresses in Railroad Track herewith presents its fourth progress report.

I.—Introduction

1.—Preliminary.—As has been the case since its organization in 1914 the Committee has co-operated with the Special Committee on Track of the American Railway Engineering Association, the membership of the committees of the two Societies being almost identical, and this report is presented simultaneously to the two Societies, and also to the American Railway Association which has furnished financial support and has otherwise co-operated in the work of the investigation.

The first progress report of the Committee was published in Vol. LXXXII of the *Transactions* of the American Society of Civil Engineers (1918); it may also be found in Vol. 19 of the *Proceedings* of the American Railway Engineering Association. The second progress report was published in Vol. LXXXIII of the *Transactions* of the American Society of Civil Engineers (1919-20), in Vol. 21 of the *Proceedings* of the American Railway Engineering Association, and as *Circular No. S-II-10* of the American Railway Association. The third progress report was published in Vol. LXXXVI of the *Transactions* of the American Society of Civil Engineers (1923), Vol. 24 of the *Proceedings* of the American Railway Engineering Association, and as *Circular No. IV-32* of the American Railway Association. In the three progress reports, comprising in all about 520 pages, the tests that have been made on railroad track and in the laboratory to give information on the action of rails, ties, and ballast under the loads of locomotives are described and the results compared and discussed. The data and discussion of a great variety of tests are to be found in the reports given in these *Transactions* and *Proceedings* of the Societies.

The work herein reported includes tests on straight track and curved track of the electrified portion of the Chicago, Milwaukee and St. Paul Railway in Montana, with several types of electric locomotives, one steam locomotive, and loaded freight cars, reported as "II.—Tests on the Chicago, Milwaukee and St. Paul Railway", and tests on straight track and curved track of four Eastern railways with steam locomotives and loaded cars, under the heading, "III.—Tests on Eastern Railroads", which were made principally to get information on the influence of canting of rail by the use of inclined tie-plates. With the great amount of material obtained in the field work it was not possible, in view of the limitations of space, to report all the data of the tests, and an effort has been made to choose for publication those parts which have the most direct bearing on the problems considered and which also may be useful for purposes that may develop hereafter.

The Committee is continuing work on the subject assigned to it.

2.—Acknowledgment.—Since the third progress report was issued, the expenses of carrying on the work of the Committee have been paid largely

from contributions made by the American Railway Association. The Committee expresses its appreciation of this support. Important contributions have also been received from companies that manufacture steel rails, these funds being handled through the American Society of Civil Engineers and the American Railway Engineering Association. The Committee expresses appreciation of the contributions made by the Illinois Steel Company, the Carnegie Steel Company, the Tennessee Coal, Iron and Railway Company, the Algoma Steel Corporation, the British Empire Steel Corporation, the Cambria Steel Company, the Bethlehem Steel Company, the Colorado Fuel and Iron Company, the Midland Steel Company, and the Westinghouse Electric and Manufacturing Company.

The co-operation of railroads in furnishing facilities for the test work has itself been an important contribution. The Chicago, Milwaukee and St. Paul Railway, C. F. Loweth, Past-President, Am. Soc. C. E., Chief Engineer, and L. K. Sillcox, M. Am. Soc. C. E., General Superintendent of Motive Power, gave excellent opportunities for the conduct of the test work by providing locomotives, track, train crews, and other facilities throughout the six weeks occupied by the tests. L. J. Murray, Special Apprentice, gave valuable assistance throughout the tests, and others of the Engineering, Mechanical, and Operating Departments were very helpful. On the Eastern work, the Baltimore and Ohio Railroad, Earl Stimson, M. Am. Soc. C. E., Chief Engineer, Maintenance, the Reading Company, R. B. Abbott, Assistant General Superintendent, the Lehigh Valley Railroad, G. L. Moore, Engineer, Maintenance of Way, and the Richmond, Fredericksburg and Potomac Railroad, E. M. Hastings, M. Am. Soc. C. E., Chief Engineer, all furnished adequate facilities for making the tests conducted on the track of these several railroads. In the tests on both the Chicago, Milwaukee and St. Paul Railway and the Eastern railroads much interest was taken and valuable assistance rendered by those in charge of operation.

The field and office work has been carried on by a regular staff well fitted for this class of work. E. E. Cress, Assistant Engineer of Tests, in charge of field and office work and the reduction of data and preparation of material, is entitled to much credit; his thorough familiarity with details, keen grasp of the problem, and helpfulness in the study and preparation of the data have made his services particularly valuable. Louis J. Larson, Assoc. M. Am. Soc. C. E., Associate in Theoretical and Applied Mechanics in the University of Illinois, by reason of his special training and insight in testing work, has given valuable service in both laboratory and field work during the summer seasons. Mr. R. Ferguson has given helpful service in the field work, the reduction of data, and the preparation of the report, as did Mr. H. L. Parr in the field work. Others have assisted in the work from time to time and all have given loyal and careful service.

The University of Illinois has continued to co-operate in the work by giving the use of laboratory, shop, and office facilities, and through the service of the members of the staff of the Engineering Experiment Station from time to time.

II.—Tests on the Chicago, Milwaukee and St. Paul Railway

3.—The Tests.—The tests on the Chicago, Milwaukee and St. Paul Railway were conducted in July and August, 1923, on the electrified section of the railroad near Lennep and Loweth, Mont., stations which are on the Rocky Mountain Division 35 and 45 miles west of Harlowton. A purpose of the tests was to obtain data by which the effects on straight track and on medium and sharp curves produced by the several types of electric locomotives run at several speeds could be judged and compared with each other and with a standard type of steam locomotive. Stresses in the rail under and between wheels and for both vertical and lateral bending of the rail were determined and various other observations bearing on the action of the rail and the locomotives were made.

4.—The Track.—The track on which the tests were conducted was the single-track main line. The test section of straight track was within the yard limits of Lennep about 2 500 ft. east of the station building and alongside a passing track. The gradient was 0.79% downward to the east. The test section of 6° curve was at Loweth near the telegraph office and a short distance eastward from the summit of the pass over the Belt Mountains. The gradient was 0.79% downward to the east. A passing track lay alongside. The test section of 10° curve was about 2 300 ft. west of the Lennep station building. Its gradient was 0.75% downward to the east. The 6° curved to the left and the 10° to the right when going eastwardly, the direction in which the test runs were made. All three test sections were on a slight fill, which was solid and well compacted.

The rail was 90-lb. A. R. A.-A. That on the straight track was laid in 1918, on the 6° curve in 1916, and on the 10° curve in 1921. The rail on the straight track and 6° curve was only slightly worn. On the 10° curve the outer rail was worn $\frac{1}{8}$ in. or more on the gauge side; there was a small lip on the inner rail. The moment of inertia of the full section about a horizontal gravity axis is 38.7 in.^{4} and that about the vertical gravity axis, 7.5 in.^{4}. The corresponding section moduli, $\frac{I}{c}$, are 15.2 in.^{3} about the horizontal axis and 2.9 in.^{3} about the vertical—both being with respect to a fiber at the base of rail. As the wear of the rail affects the value of the section modulus with respect to the base of rail only slightly, the values of $\frac{I}{c}$ for the full rail were used in the calculations to determine bending moments from stresses in rail. The splice-bars were 24-in., heat-treated angle-bars of A. R. A.-A. design.

The tie-plates were $6\frac{1}{2}$ by 9 in. on the straight track and 10° curve, and $6\frac{1}{2}$ by $8\frac{1}{2}$ in. on the 6° curve. The eccentricity of the former (distance from center of length of tie-plate to center of base of rail) was $\frac{3}{16}$ in. and of the latter, $\frac{5}{16}$ in. The tie-plates were approximately flat.

The ties were 7 by 8 in. by 8 ft. untreated fir and except for renewals had been in the track eight years. The ties at the test locations were in fairly good condition except a few on the 6° curve. On the straight track the spacing was 18 to the rail length of 33 ft., and 20 on the two curves.

The ballast was mainly gravel. On straight track it extended 17 in. below the bottom of the tie and on the 10° curve, 15 in. On the 6° curve there were 12 in. of mixed gravel, cinders, and sand. In all cases the lower part of the ballast was good but the upper part was in poorer condition.

The maintenance work for the season of 1923 had been deferred until after the tests were made so as not to change conditions, and the track was thus not up to the usual standards of the railroad. Generally it was in only fair surface and alignment. At the test location on straight track the alignment was fairly good, though on each side there were stretches with poor alignment. The surface was somewhat uneven. In spots tamping was needed. A number of the tie-plates next to the joints were found to be somewhat loose. The 6° curve, a point where the speed of trains was generally slow, was in poorer condition, the inner rail being uneven and a number of the tie-plates under that rail being loose. At the test section the alignment was good. The 10° curve was in very good condition, holding close to a 10° curvature, except that the alignment over a pile bridge 200 ft. west of the test section was poor and that of the easement at the beginning of the curve was not good; other than at these points the locomotives rode well on this curve to a point beyond the test section. The condition of the track in the three locations is described at this length in order that it may be judged whether conditions of the track over the approach to the stretches at which the instruments were located would be likely to affect the action of the locomotives and cars in their passage over the test sections, particularly at the higher speeds.

The gauge on the test section and adjoining it was 4 ft. 8½ in. on straight track, 4 ft. 8⅞ in. on the 6° curve, ranging from 4 ft. 8⅝ in. to 4 ft. 9⅛ in. on the 6° curve, and 4 ft. 9 in. on the 10° curve with little variation. The superelevation of the outer rail of the 6° curve was 3.2 in., corresponding to a speed of 29 miles per hour, and there was little variation from this value. On the 10° curve the superelevation was 3.9 in., corresponding to a speed of 24 miles per hour. The maximum speeds used in the tests, 50 miles per hour on the 6° curve and 40 miles per hour on the 10° curve, were more than one and two-thirds times the speeds corresponding to the superelevation.

The data from measurements of the depression of the rail made under the weight of a loaded car and of a light caboose and used in the calculation of the value of the modulus of elasticity of rail support, u, by the method described in previous progress reports, gave an average value of 1 100 lb. per in. for the straight track.

5.—The Locomotives and Cars.—Three principal types of electric locomotive were in regular use on the electrified portions of the railroad: (*a*) the General Electric freight locomotive; (*b*) the General Electric passenger locomotive; and (*c*) the Westinghouse-Baldwin passenger locomotive. The last type was represented in the tests by three forms, the original form of the locomotive and two modifications. The five electric locomotives used in the tests then represented all the types of motive power used for main-line service on the electrified divisions of the railroad. The steam locomotive was represented by one of the Mikado type. The General Electric freight locomotive

has been described in detail.* A description of the General Electric and Westinghouse-Baldwin passenger locomotives of the Chicago, Milwaukee and St. Paul Railway may be found in two articles published by the American Institute of Electrical Engineers.† These articles and information received from the railroad company form the principal basis for the descriptions which follow.

The three principal types of electric locomotive differ widely in both the mechanical and the electrical features of their design. A common characteristic was that the axles of the drivers were individually driven, separate motors being applied to the driving axles, and there were no connecting rods or side rods for which counterbalancing must be provided, and, therefore, no stresses in the track due to imperfect counterbalance. All the electric locomotives were symmetrical with respect to the center of their length.

The power supply was direct current of 3 000 volts.

In Figs. 1 and 2 are diagrams of the locomotives—wheel loads and spacings and other data. A diagrammatic representation of the equalization system is given in Figs. 3 and 4. Additional data will be found in Table 1.

The General Electric freight locomotive, 10221, had been in freight service since 1916; it does not differ from the other freight locomotives in use on the electrified divisions. The characteristics of this type, as affecting the action on the track, are the unusually long wheel spacings, the great over-all length, and the fact that a spring gear drive with axle suspension motors is used. The suspension of the motors puts on each driving axle an unsprung weight of 16 250 lb., a greater unsprung load per axle than is found in either of the newer types. Each half of the locomotive has a four-wheel guiding truck similar to the ordinary type used on steam locomotives, and two trucks, each containing two pairs of drivers. There is a hinged connection between the two halves, which also permits some vertical movement, but no horizontal movement other than a swing. The two sections of the cab are supported on bolsters set midway between pairs of driving axles; the center plate on the bolsters allows longitudinal and pivoting movement, but no lateral displacement of the cab.

The General Electric passenger locomotive, 10254, is of the type used between Othello and Tacoma, Wash., and had been in service since 1920. The locomotive is driven by bi-polar gearless motors having the armatures wound on the axles. Other characteristics of the locomotive, as affecting the track, are the large number of drivers, which are very closely spaced, and the relatively small diameter of the drivers and the relatively small loads they carry. The close wheel spacing has an important effect in the development of smaller stresses in the rail than would occur with a longer spacing. The first two of the twelve drivers on one side are grouped in a truck frame with a guiding wheel, then follows a unit of four drivers; the second half is the reverse of this. The running gear is thus articulated in three places. The cab is divided into three sections and supported in an unusual manner. The end sections contain auxiliary equipment, switches, resistances, and the com-

* *General Electric Review*, November, 1916, p. 929.

† *Journal*, Am. Inst. Elec. Engrs., April, 1920.

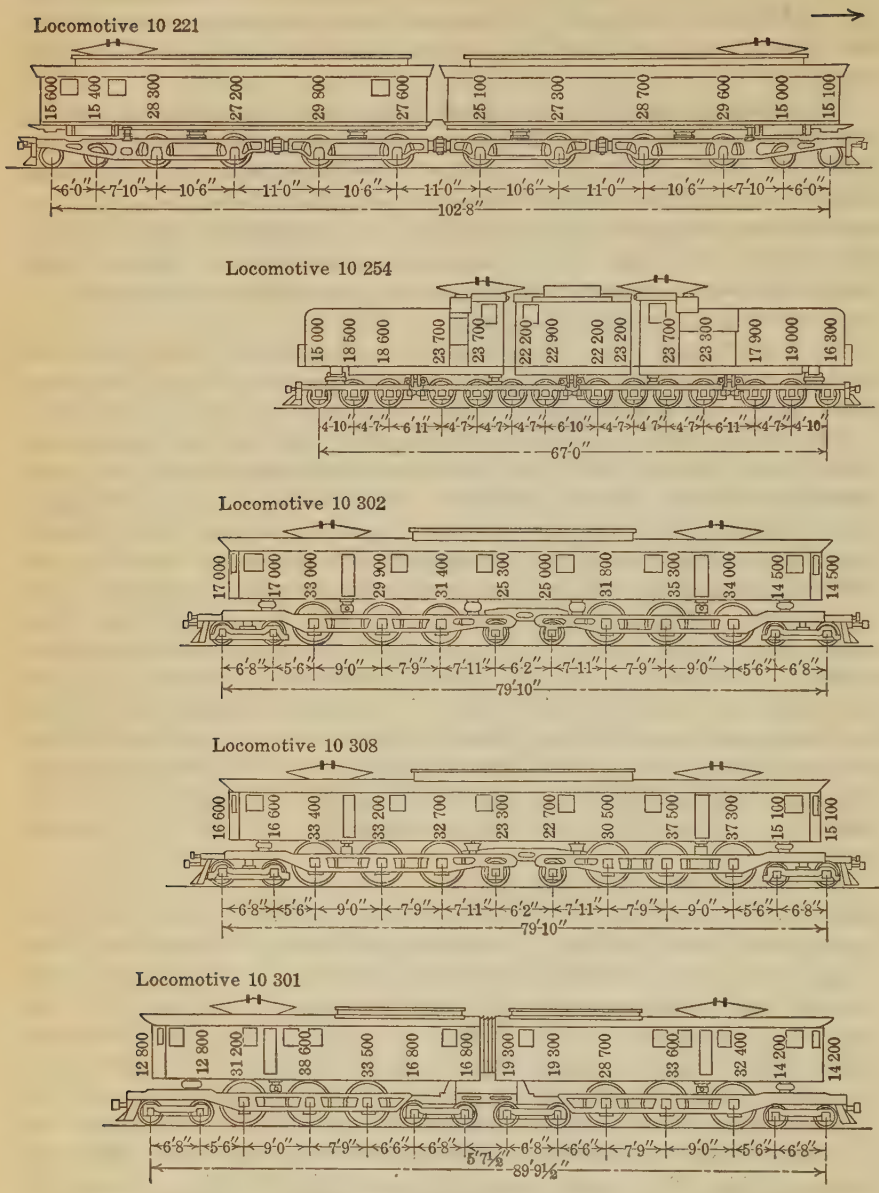


FIG. 1.—DIAGRAMS OF ELECTRIC LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

partments for the crew. These end sections are fastened rigidly at the center of each middle set of drivers and rest on rollers bearing on inclined planes over the end sets of the running gear. The center section containing the train heating and lighting equipment is suspended from four brackets on the adjacent ends of the other two sections and has no other connection with the remainder of the locomotive to give it support. The guiding axle at either end of the locomotive is allowed some lateral play but the movement is damped by wedges placed above the journal boxes. Other details bearing on the action of the cab and running gear in traversing curves can not well be given. Having the motor armatures on the axles places a considerable proportion of the total axle load below the springs; the results of the tests thus have some bearing on the problem of the effect of the unsprung weight on the track. The unsprung or dead weight per driving axle is given as about 9 600 lb. In order to have one of this type for use in the tests, this locomotive was brought from the Coast Division, a distance of 600 miles, including the intermediate non-electrified divisions of the railroad.

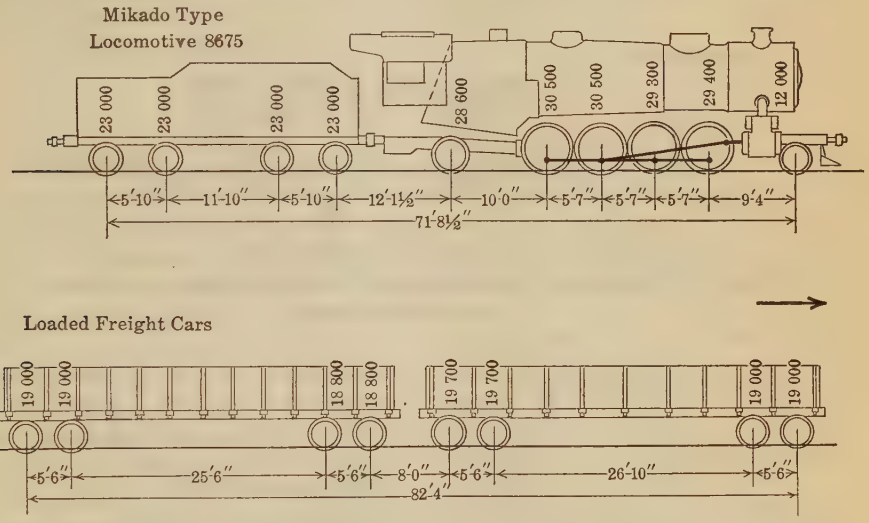


FIG. 2.—DIAGRAMS OF STEAM LOCOMOTIVE AND CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The Westinghouse-Baldwin locomotive, 10302, is of the type being operated between Harlowton, Mont., and Avery, Idaho, and had been in service since 1920. This locomotive is essentially two Pacific running gears coupled back to back, supporting a single cab that extends the whole length of the locomotive. It has "Woodward" four-wheel guiding trucks and "Rushton" trailers. The coupling between halves of the running gear consists of a long bar with a pin at each end. The cab is supported by six spring plungers on each half of the running gear, two being in line with the center pin and two at each end. The driving torque is transmitted from twin armature motors (this weight being spring supported) to a quill drive and thence to the individual driving axles. Since they were put into service, the weight of all the Westinghouse-Baldwin locomotives has been increased by the substitu-

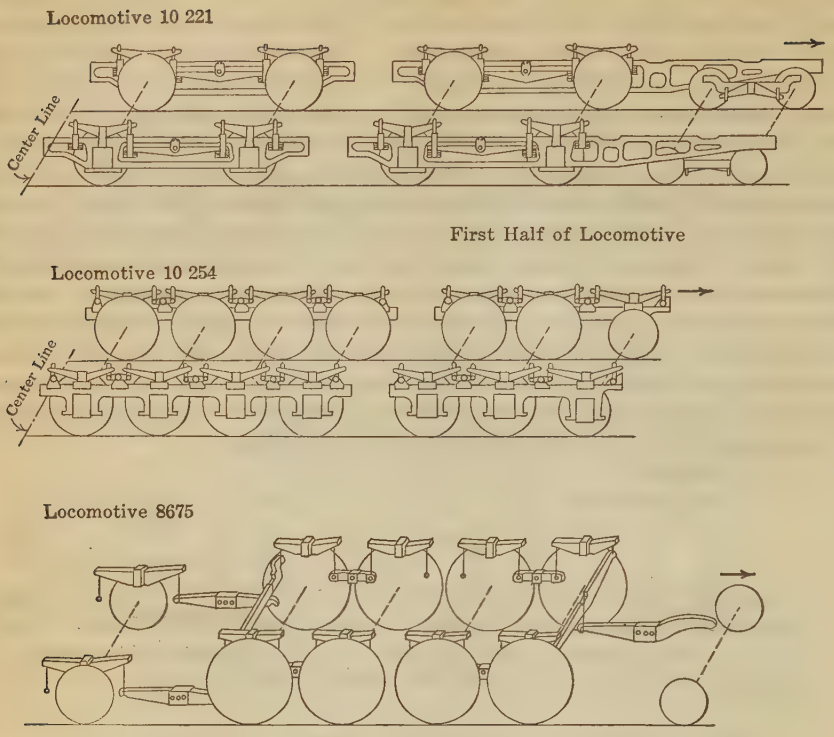


FIG. 3.—DIAGRAMMATIC REPRESENTATION OF THE EQUALIZATION SYSTEMS OF THE LOCOMOTIVES.

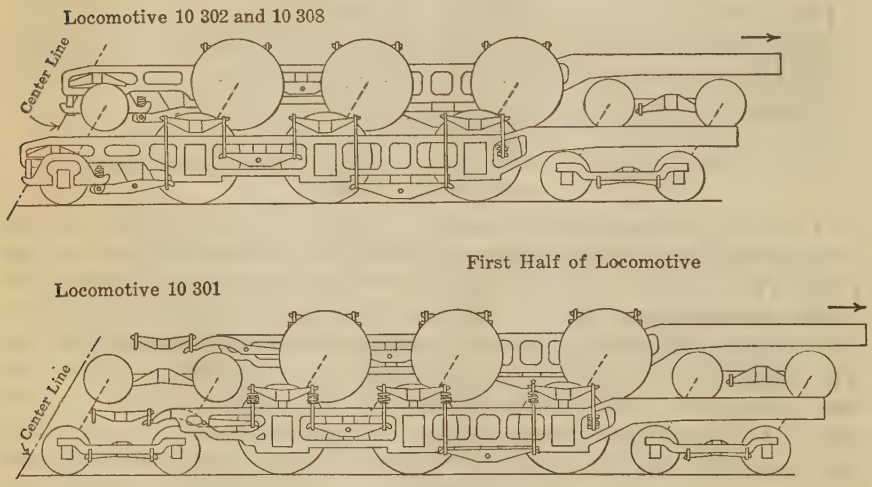


FIG. 4.—DIAGRAMMATIC REPRESENTATION OF THE EQUALIZATION SYSTEMS OF THE LOCOMOTIVES.

tion of heavier frames, by additional bracing for the cabs, and by changes in the quill drives, so that the wheel loads reported in the diagrams and tables are somewhat higher than those named in the earlier descriptions of the locomotives.

TABLE 1.—Data of the Three Types of Electric Locomotives of the Chicago, Milwaukee, and St. Paul Railway.

	General Electric Freight Locomotive 10 221.	General Electric Passenger Locomotive 10 254.	Westinghouse-Baldwin Passenger Locomotive 10 302.
Total weight of locomotive, in pounds	569 225	547 700	617 500
Total weight on drivers, in pounds	447 100	485 300	390 700
Average load per driving wheel, in pounds	28 000	20 200	32 600
Unsprung weight per driving wheel, in pounds	8 100	4 800	3 900
Diameter of driving wheels, in inches	52	44	68
Length over all, in feet	112.0	76.0	88.6
Height of center of gravity, in inches	62.6	60	71.5
Number of motors	8	12	6*
Total horse-power, 1-hour rating	3 440	3 500	4 200
Total horse-power, continuous	3 000	3 200	8 400
Total tractive effort, continuous, in pounds	70 700	42 000	49 000
Corresponding speed, in miles per hour	15.9	28.4	26.0

*Twin armature motors.

Modifications had been made in two of the Westinghouse-Baldwin locomotives in an effort to find whether improved lateral flexibility and gain in maintenance would result, and these two locomotives were also used in the tests. In Locomotive 10308 the cab was given a different support, the ends being held on a long vertical link or pendulum; a different connection was made between the two halves of the running gear, but the "Rushton" trailers were allowed to remain. The cab of Locomotive 10301 was divided into two parts, and the over-all length increased about 10 ft. Two four-wheel trucks were placed in the middle, replacing the trailers or forming two sets of double trailers. A link was placed in this truck of the second half of the locomotive to improve its guiding action. The addition of four wheels decreased the weight on the drivers somewhat.

Locomotive 8675 is a Mikado type freight locomotive known as the heavy U. S. Railroad Administration design (Type L 3 of the Chicago, Milwaukee and St. Paul Railway) and had been in service since 1919. It had been run 20 000 miles since shopping. It was brought in from the Musselshell Division at Miles City, Mont., for use in the test. The weight on a trailer is nearly as great as that on a driver. The total load on the drivers* was 239 000 lb. The same design is in use on railroads in various parts of the United States.

In comparing the locomotives, it is seen that the driving wheel base was 10 ft. 6 in. on the General Electric freight locomotive, 13 ft. 9 in. on the General Electric passenger locomotive, and 16 ft. 9 in. on the Westinghouse-Baldwin passenger locomotives and the Mikado type. Even the last two are

* The unsprung weight, including the wheel and axle and bearings, a proportion of the connecting rod and side-rods, but not the effect of counterbalance at speed, is estimated to be 6 000, 6 500, 10 000, and 6 000 lb., respectively, for a first, second, third, and fourth driver.

considerably shorter than the wheel base of 22 ft. found with Santa Fé type locomotives. The diameter of the drivers of the General Electric freight locomotive was 52 in., that of the General Electric passenger locomotive, 44 in., that of the Westinghouse-Baldwin locomotive, 68 in., and that of the Mikado type, 63 in. The locomotives were taken direct from service without change or special preparation. All of them may be said to have been in ordinarily good condition mechanically.

Although the electric locomotives were designed to run in either direction, for the tests generally the test runs were made eastward with the following named end in front: 10221, B; 10254, A; 10302, No. 2; 10308, No. 1; and 10301, No. 2. It was the practice to turn some of these locomotives at the end of each run for a certain mileage, then to reverse their direction for a time.

The freight cars were Chicago, Milwaukee and St. Paul Railway coal cars that had seen much service. They were loaded with coal so as to give nearly equal weights on the two ends. The weight was taken for each truck of the loaded car and then the whole car was weighed as a check. The wheel loads given in Fig. 2 are one-fourth of the load on one end of the car. The wheels were 33 in. in diameter, and the wheel base, 5 ft. 6 in.

6.—Conduct of Tests and Reduction of Data.—The methods used in the tests of track were generally the same as those used in the tests described in the previous progress reports. Eight stremmatographs were used simultaneously. These instruments had been rebuilt with some minor improvements over the form described in the first progress report.* Four stremmatographs were placed on one rail between ties at distances apart of about 6 ft., and the other four instruments directly opposite on the other rail. The driving mechanism used rotated the disks of all the instruments simultaneously. The correlation of a point on the record of one of the disks with the point of any other disk at the same moment was possible. As the locomotive passed the test section a record of the strains in the rail was made on each instrument. The passage of each wheel of one side of the locomotive thus was recorded on four instruments, and each wheel of the other side on four other instruments. As in each instrument one disk recorded the strain at one edge of the base of rail and another that at the other edge, the differences in the stresses at the two edges, and, therefore, the lateral bending stresses, were obtainable. Three or four runs were recorded on a disk, according to the length of the locomotive. The disks were then taken out of the instruments and fresh ones inserted.

Except in the motoring-regenerating tests made with two locomotives in action, the power was shut off as the locomotive approached the test section of track and the locomotive "coasted" past the instruments. The speeds were read from a speedometer in the cab connected with the tread of a wheel, the instrument having been checked up over a measured length of track. The locomotives were easily controlled to give the speed desired. After a run, the locomotive was backed over the track and the next run made. The speeds used

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1224.

in the tests ranged from 5 to 60 miles per hour on straight track, 5 to 50 miles per hour on the 6° curve, and 5 to 40 miles per hour on the 10° curve. The sequence was usually 5, 25, 40, and 60, or 5, 30, 40, and 50, or 5, 25, and 40 miles per hour. About sixteen runs were made at each test location for each speed with the General Electric freight locomotive, the General Electric passenger locomotive, and the original form of the Westinghouse-Baldwin locomotive, and about eight runs with the other forms of Westinghouse-Baldwin locomotive and the Mikado type locomotive and cars.

Fig. 5 shows the position of the stremmatographs in one of the tests of curved track. Variations from this arrangement were made, but the diagram is representative of all the tests. As the track was laid with alternate joints it was necessary that a rail joint lie within the test section.

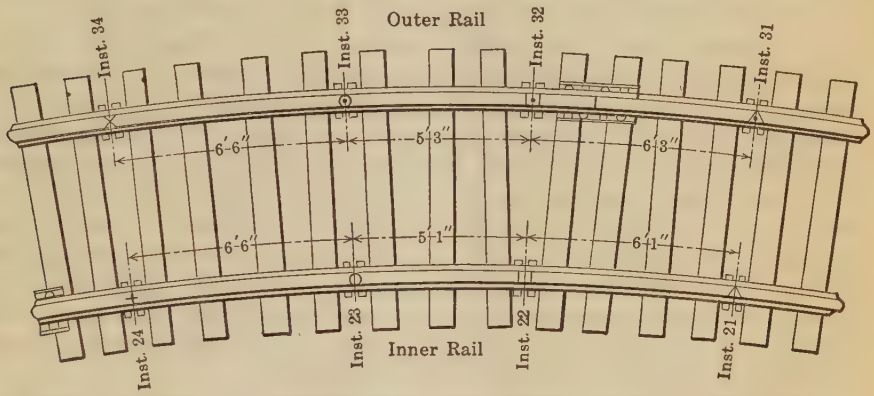


FIG. 5.—POSITION OF THE EIGHT STREMMATOGRAPHS IN A TEST OF CURVED TRACK.

The process followed in the reduction of the data obtained by the stremmatographs was the same as that which was described in the earlier progress reports. The stremmatograph records were read with a binocular microscope fitted with a micrometer eyepiece. Readings were taken for points in the record corresponding to a wheel over the instrument and to a point between wheels, the high point of the record being assumed to have been made when the wheel was directly over the instrument. In reducing these measurements the readings were multiplied by the proper microscopic constant and then reduced to stresses by multiplying by a constant which involves the position of the neutral axis of the rail section, the vertical distance of the needle-bar below the base of rail, the modulus of elasticity of steel (taken as 30 000 000 lb. per sq. in.), and the gauge length (which was 4 in.). A correction was also made to allow for the variation of moment and stress over the gauge length to obtain the maximum stress at the middle of the gauge length by multiplying the average stress over the gauge length for wheel over instrument by the factor, 1.04, as was done with the tests recorded in the first progress report, so that the stresses reported at points of positive moment are stresses in pounds per square inch at the base of the rail at the middle of the gauge length. The variation over the gauge length at points of

negative moment was slight and no correction was used for the stresses at such points.

The records on the disks were found to be unusually good, that is, readable and definite. As must be expected some of these were indefinite and not usable. On the straight track, those obtained at all but the speed of 60 miles per hour generally were clear, probably 85% were usable. At the speed of 60 miles per hour, the proportion available was much less, the jar from the rail affecting the clearness of the record; particularly for the two instruments on one rail adjacent to the joint less than half the records were usable. On the 6° curve the proportion of usable records at 50 miles per hour was considerably less than at the lower speeds, and the outer rail gave better records than the inner rail. The records on the 10° curve were generally excellent at all speeds, the proportion of usable records being about 95 per cent.

All readable records were used in making up the averages even though the corresponding record on the companion disk was defective. In calculating stresses in rail on straight track, instead of first taking the average of the individual stress at the two edges of the rail, the average of all usable stresses at one edge of the base at a given speed was calculated, and also the average at the other edge, it having been found that for good records the results are nearly the same as those obtained by first averaging the individual values at the two edges. In calculating the ratio of stress at outside edge to mean stress in base of rail, individual ratios were desired and the only values used were those having usable records for the two edges made at the same time. On curved track, average values were calculated for each edge of base of rail separately, ratios for the two edges not being found.

The accuracy of the records and their reduction is considered to be at least as good as that of the stremmatograph data reported in the previous progress reports, say, 800 lb. per sq. in. for an individual record and perhaps 300 lb. per sq. in. for the averages.

The time required to make the 320 000 microscopic measurements of the records involved in the Western work was considerable, although with clear records skilled observers made 1 200 such readings in a day.

On the curved track, measurement of the lateral movement of the inner and the outer rail, as the locomotive moved by at a speed of about 2 miles per hour, was made. A wooden bar with an Ames dial gauge at one end had one end attached to a fixed point 5 ft. from the rail and the other against a point on the head of the rail, the dial being read as the wheel passed and at a point midway between wheels. Measurement was made of the position of the flanges of the wheels of the locomotive after it came to rest on the two curves without the application of brakes, and also of the alignment of the axles and the amount of the journal play under the same condition. The contour of the tires and of the rails was also obtained.

During the latter part of the time given to the field work a method was developed for determining the position of the bearing of the wheel on the rail as it passed by at any speed. A copper wire placed under one rail and over the other was kept taut and pulled a distance of about 6 in. between the

passage of one wheel and the next, one man pulling the wire and another at the other side of the track keeping steady tension in the wire. Near the end of the field tests, it was found that a series of punch marks in a line along the head of the rail would give impressions on the wire that enabled the position of the flattened wire to be determined accurately with respect to the width of the head of the rail and thus the bearing of wheel on rail to be known. If the flange of the wheel passed quite close to the rail an impression by it was also made. The wire used was that available; the diameter ranged from 0.06 to 0.10 in.

7.—*Results of Tests on Straight Track.*—The tests on straight track give information on the stresses in rail developed by the six types of locomotives with their great variety of wheel spacings and loadings, of positions of load, and of arrangement of springs and equalizers; they also provide a basis for a comparison with the results on curved track.

In Tables 2 and 3 are given the values of the mean stresses in base of rail under each wheel at three or four speeds for the five electric locomotives, each value being the average of the records of the eight instruments on the two rails. The mean stresses in base of rail developed by the Mikado type locomotive are given in Table 4. In Figs. 6 to 12 the values of the mean stress in base of rail are plotted to scale and also the mean stress for points between wheels. It should be noted that each value is the average of a large number of observations and that individual observations will be either greater or less than the average value.

It will be seen that the stresses developed in the 90-lb. rail under the drivers at a speed of 5 miles per hour average about as follows: General Electric freight locomotive (10221), 17 000 lb. per sq. in.; General Electric passenger locomotive (10254), 8 000 lb. per sq. in; Westinghouse-Baldwin passenger locomotives (10302, 10308, and 10301), 15 500 lb. per sq. in., ranging from 13 500 to 19 700 lb. per sq. in. under the several drivers; and Mikado type locomotive (8675), 12 000 lb. per sq. in., varying from 10 000 to 14 000 lb. per sq. in., under the several drivers. With the exception of the General Electric passenger locomotive, which developed smaller stresses in the rail, these stresses are within the general range of stress noted in the previous progress reports for the heavy steam locomotives with 90-lb. rail on straight track at the low speeds. In the discussion of the effect of speed on stress in rail that is given in Article 8 which follows, it is shown that a change of speed from 5 to 40 and 60 miles per hour results in an increase of stress in rail that is generally less than was found in the tests previously reported.

In the first progress report a method* was developed for calculating the stresses in the rail produced by any arrangement of static wheel loads on a track with a given rail and known stiffness of track. The effect of individual wheels, both for the rail under the wheel and for points away from the wheel, was determined from a formula which involved the wheel load, the moment of inertia of the rail, and the stiffness of the rail supports (ties and ballast) and from a master diagram that shows the change in bending

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1202; *Proceedings*, Am. Ry. Eng. Assoc., Vol. 19, p. 882.

TABLE 2.—Mean Stress in Base of Rail on Straight Track with General Electric Locomotives of the Chicago, Milwaukee and St. Paul Railway.

(Stresses are given in pounds per square inch.)

Speed, in miles per hour.	Rear End Truck.		Driver.								Head End Truck.	
	2.	1.	8.	7.	6.	5.	4.	3.	2.	1.	2.	1.
						Locomotive 10 221						
5	9 900	6 900	17 100	16 800	17 100	17 400	16 800	16 900	17 000	16 200	7 400	9 800
25	10 300	7 200	17 200	17 400	17 200	17 600	16 900	17 300	17 100	16 700	7 700	10 400
40	10 600	7 600	18 000	17 900	18 500	18 300	17 400	18 000	17 700	16 800	7 900	10 800
			12.	11.	10.	9.	8.	7.	6.	5.	4.	3.	2.	1.		
							Locomotive 10 254									
5	8 100	.....	7 400	8 000	8 700	7 600	7 800	8 900	7 900	7 400	7 400	8 700	8 000	7 700	.....	8 900
25	7 900	.....	7 200	7 500	8 400	7 300	7 600	8 700	7 600	7 400	7 300	8 500	7 700	7 400	.....	9 000
40	8 500	.....	7 700	8 100	8 500	7 700	8 000	8 500	8 200	7 800	7 400	8 700	8 400	7 700	.....	8 700
60	9 200	.....	9 200	9 200	10 000	9 300	9 200	9 500	10 000	10 000	9 100	9 400	9 400	8 800	.....	9 300

moment away from the wheel. As an illustration of the effect of wheel spacing and load, calculations of stresses have been made by this method for Locomotives 10221 and 10254, and the values of the calculated stresses in the rail have been plotted as shaded lines in Figs. 6 and 7 beside the observed stresses at 5 miles per hour. A comparison of the two sets of values shows that the stresses and their distribution are quite similar, probably as nearly the same as are the nominal and the real weights. For Locomotive 10254, the calculated stress at points between wheels, for negative moments, is somewhat greater than the observed stress when the space between wheels is large; a smaller but opposite effect is found for the closer spacing. The two diagrams (Figs. 6 and 7) call attention to the smaller stresses that are found with smaller individual wheel loads and a favorable spacing of the wheels, a wheel load of 28 000 lb. in Locomotive 10221, producing a stress of 16 000 lb. per sq. in. and one of 22 000 lb. in Locomotive 10254 producing only 8 000 lb. per sq. in. The stress under the drivers of Locomotive 10254 is only about 50% of that due to an isolated wheel load, while that under the drivers of Locomotive 10221 is 85 per cent. As noted in the first progress report, the effect of wheel spacing will depend somewhat upon the weight of rail and stiffness of track. It is evident that wheel spacing and distribution of load are among the factors that need consideration in locomotive design.

TABLE 3.—Mean Stress in Base of Rail on Straight Track with Westinghouse-Baldwin Passenger Locomotives of the Chicago, Milwaukee and St. Paul Railway.

(Stresses are given in pounds per square inch.)

Speed, in miles per hour.	Rear End Truck.		Driver.			Trailer.		Driver.			Head End Truck.	
	2.	1.	6.	5.	4.	2.	1.	3.	2.	1.	2.	1.
Locomotive 10 302												
5	10 500	6 300	13 800	15 300	15 200	10 700	7 800	13 900	19 700	13 500	5 300	9 300
25	10 700	7 000	14 100	15 500	15 300	10 700	8 200	14 000	20 500	13 800	5 500	9 900
40	11 500	7 400	14 800	16 100	15 800	11 200	8 800	14 500	20 800	14 800	6 600	10 700
60	13 900	10 100	17 000	18 100	17 600	13 500	11 200	16 400	22 100	15 700	8 600	11 400
Locomotive 10 308												
5	9 200	5 800	14 000	14 700	14 900	9 600	8 200	15 100	16 300	15 300	6 200	10 000
40	9 800	6 700	14 800	16 000	15 400	10 200	8 500	15 300	16 500	14 200	6 600	10 700
60	10 200	7 700	15 600	17 500	16 200	10 800	9 400	16 800	17 600	14 800	8 000	11 300
Locomotive 10 301												
5	8 200	5 200	14 100	15 900	15 600	7 000 5 900	5 800 6 400	16 300	14 400	14 100	5 800	9 800
40	8 300	5 500	14 200	15 700	15 800	6 900 5 700	6 600 6 200	15 500	14 100	14 000	6 100	9 500
60	8 700	6 500	16 100	17 100	16 600	7 600 6 700	7 100 6 900	16 900	16 000	14 800	6 900	10 200

The method of calculating stresses referred to has been found to apply well to all track used in the various tests made by the Committee. The reversed process has been found to be a useful way of ascertaining the wheel loads, or, at least, of checking reported weights, especially as scales for weighing individual loads are not common. As was described in the third progress

report,* an algebraic equation is written out for the vertical bending stress for each driver (or, instead, the equivalent single wheel load); this equation involves the wheel load for a given wheel and those of the adjacent wheels, all represented as unknown quantities, and the stress in the rail under the given wheel as a known quantity. There will then be as many equations and unknown quantities as there are wheels. The solution of these equations is tedious, but not troublesome. The values found for the individual wheels may be changed proportionately to make the sum of the loads agree with the total load of the locomotive, but the correction so made has generally been within 4 per cent. In Fig. 13 are given the nominal weights and the loads calculated from the stresses in rail by this method. The agreement between the weights reported and the calculated wheel loads for 5 miles per hour is generally fairly good. In three of the locomotives a difference of more than 6 000 lb. in a wheel load was noted, with several variations at other wheels ranging from 2 000 to 4 000 lb. Generally, the calculated load on the truck wheels and trailers was greater than the reported load.

TABLE 4.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK WITH MIKADO TYPE LOCOMOTIVE OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

(Stresses are given in pounds per square inch at base of rail.)

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver.				Front truck.
			4.	Main.	2.	1.	
5	Mean value	15 000	14 100	12 400	9 700	12 600	8 400
25	Mean value	15 500	14 900	13 300	10 000	13 200	9 100
Increase in stress due to speed in percentage of stress at 5 miles per hour		3	6	7	3	5	8
40	Up	.....	10 500	15 500	8 500	10 000	.....
	Down	.....	20 500	11 500	12 500	17 500	.....
	Mean value	16 200	15 600	13 400	10 500	13 700	9 700
Increase in stress due to speed in percentage of stress at 5 miles per hour		8	11	8	8	9	16
Increase in stress due to counterbalance in percentage of stress at 5 miles per hour		.....	35	16	19	30	.....

In connection with a discussion given subsequently it should be noted that at the place where the tests on straight track were made there was a siding at one side of the main track and with this and the accompanying conditions the support possibly was somewhat different for the two rails. However, on the whole, there was found to be little difference in the action of the two rails; the summation of the stresses in the two rails for all the wheels of the six locomotives at a speed of 5 miles per hour differed only 2 or 3 per cent. Generally, too, the stresses under wheels of the same axles did not differ greatly, indicating that the axle load was evenly divided between the wheels. In a few cases the stress under one wheel was 10 to 15% greater than the mean of the stresses for the companion wheel. The two loaded cars gave stresses uniformly higher on the left rail by about 8% of the mean of

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1013; *Proceedings*, Am. Ry. Eng. Assoc. Vol. 24, p. 397.

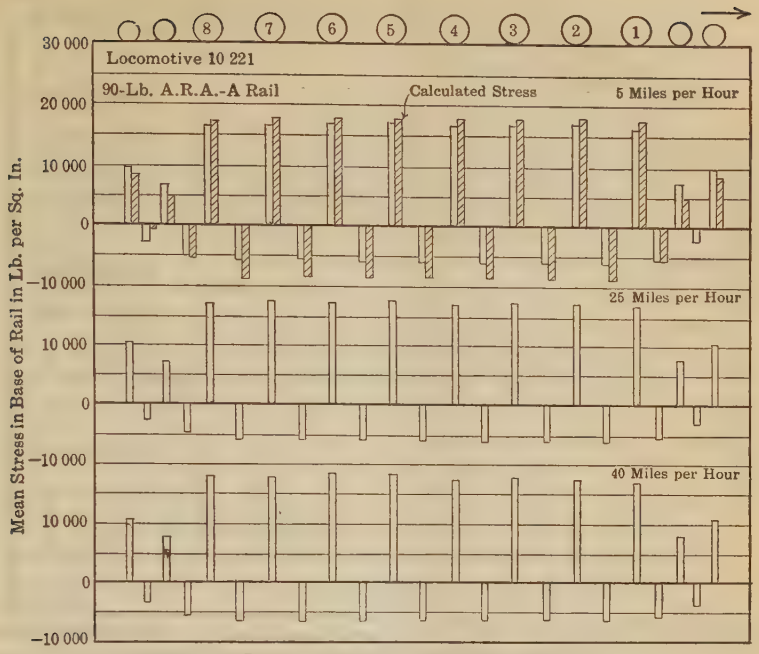


FIG. 6.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10221 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

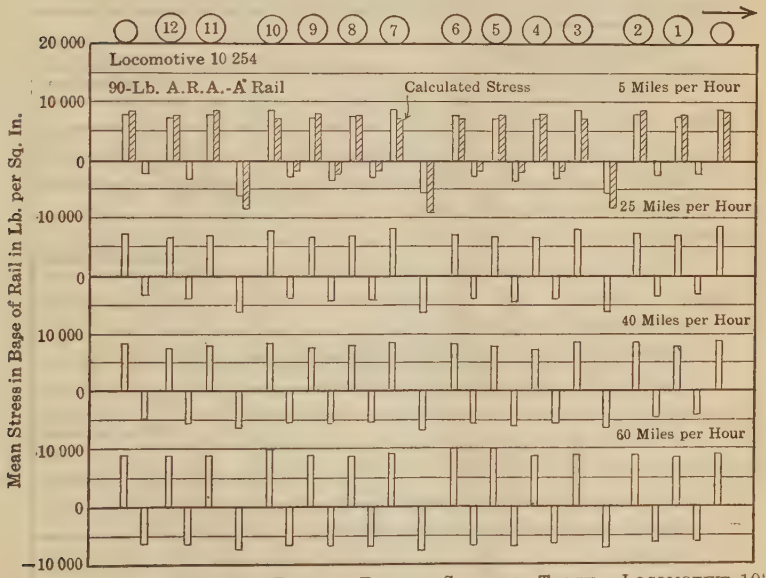


FIG. 7.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10254 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

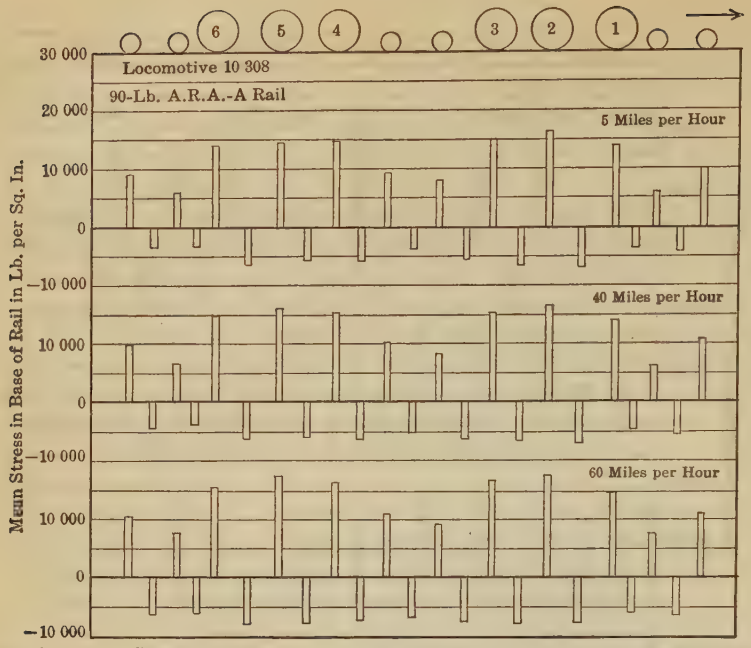


FIG. 8.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10308 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

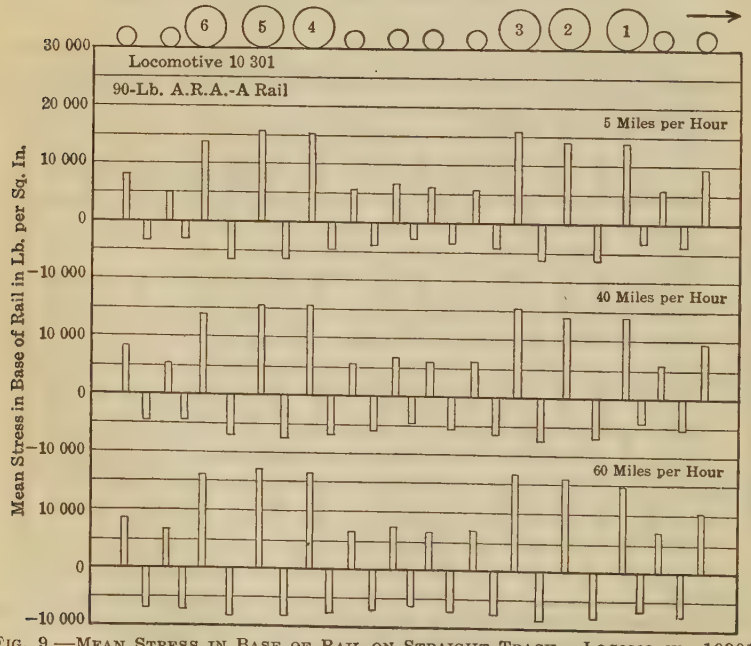


FIG. 9.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10301 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

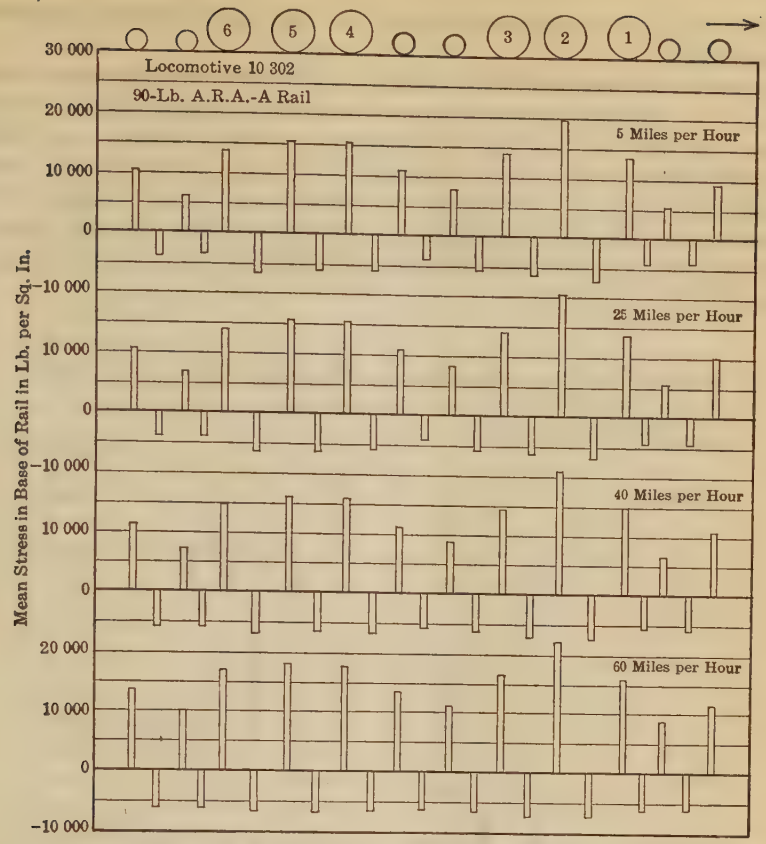


FIG. 10.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10302 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

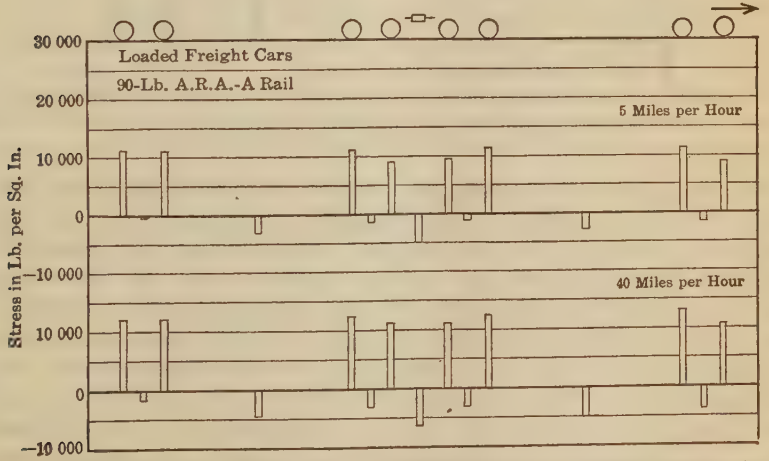


FIG. 11.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOADED FREIGHT CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

the two rails, indicating that the load was not equally divided between the two rails.

8.—Effect of Speed on Stress in Rail.—As in the previous progress reports, the ratio of the mean stress in base of rail at given speed to that found at 5 miles per hour will be taken to represent the effect of speed. In Table 5 are given general values of the increase in this ratio, which are the averages for all the wheels of a locomotive or the two cars. For 25 miles per hour the change in stress for the electric locomotives was small, hardly within the limits of accuracy of the observations, and values of the ratios were even less than unity in some cases.

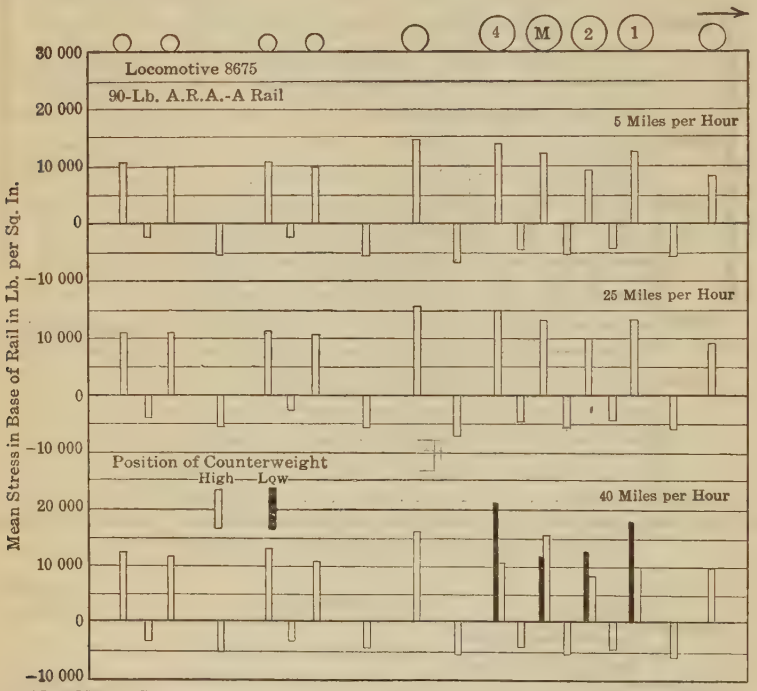


FIG. 12.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 8675 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The increase in stress on straight track due to speed is seen to be relatively small, indicating excellent general design in the locomotives. The effect of speed for the freight cars also is small. The values of the ratios for the locomotives and loaded freight cars are plotted in Fig. 14. It is apparent that on straight track the effect of speed is greater than is indicated by the first power of the speed and that a change in rate of increase occurs beyond a speed of 30 miles per hour. The values of the mean stresses in base of rail on straight track are also plotted in Fig. 14. Comparison of the individual electric locomotives can not well be made; especially in the case of Locomotive 10254 are the changes in stresses in rail so small as to involve questions of the significance of a rate of increase of stress. It will be noted that the

stress in rail under the wheels of this locomotive increases from 8 000 lb. per sq. in. at 5 miles per hour to 9 400 lb. per sq. in. at 60 miles per hour, the increase in stress being small and the stress at 60 miles per hour being much less than that found under the other locomotives.

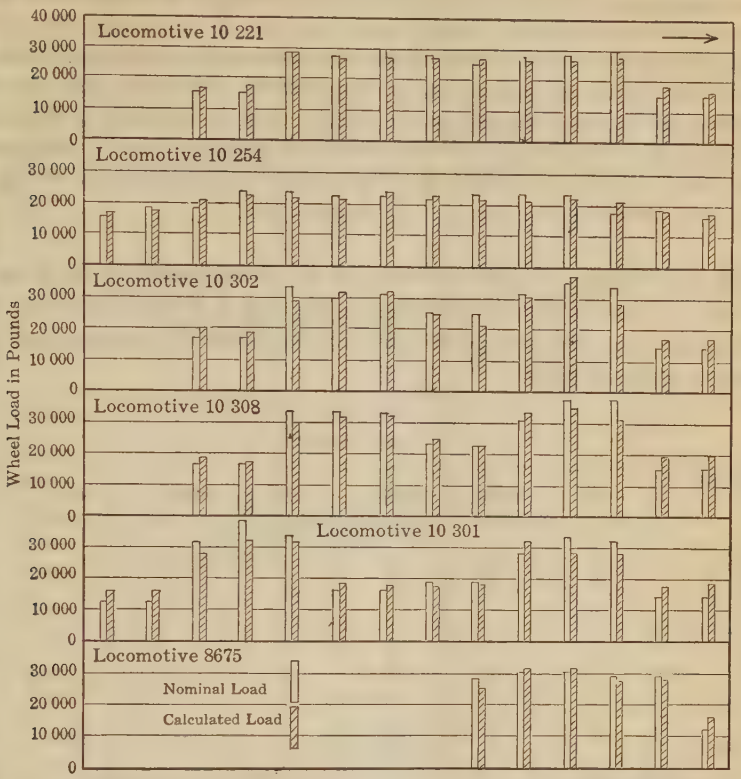


FIG. 13.—REPORTED NOMINAL WHEEL LOADS AND CALCULATED WHEEL LOADS FOR SIX LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The values already given are averages for the two rails. For the straight track the increase in stress at 60 miles per hour with Locomotive 10254 is greater in the right rail than in the left; for the other electric locomotives it is greater in the left rail, a statement that applies also to the Mikado type locomotive (8675) at 40 miles per hour. The increase for the rail having the greater change is from one-fifth to two-fifths more than the average percentage increase for the two rails. It is evident that the conditions of position and support for the two rails at the test section were such as generally to develop greater speed effect in the left rail than in the right. It is also true that the stresses in the right rail with Locomotive 10254 running at 5 miles per hour are less than those in the left rail (7 500 lb. per sq. in. in the right rail and 8 500 lb. per sq. in. in the left rail), and the stresses in the two rails at 60 miles per hour are nearly the same (9 300 and 9 500 lb. per sq. in., respectively). The records for the several instruments on the same rail vary

also one from another, showing that the effect of speed differs from point to point along the rail.

It was found that although the effect of speed on straight track differed considerably for different wheels, the larger percentages of increase were mostly on wheels having a relatively small load, as, for example, the wheel of a leading truck, which thus produced a small stress relatively to that under other wheels, the resulting stress at the higher speed itself generally not being greater than that of other wheels of its kind in the locomotive. Generally, too, the wheels developing the highest stresses at the high speeds do not have a large speed effect; thus, the stress in the left rail under the middle driver of the first unit of Locomotive 10302 at a speed of 60 miles per hour (24 000 lb. per sq. in.), is only 14% greater than that at 5 miles per hour.

TABLE 5.—General Values of Increase in Mean Stress in Base of Rail at a Given Speed Over the Mean Stress at 5 Miles Per Hour.

(The increase is the average for all wheels and is given in percentage of the stress at 5 miles per hour.)

Speed, in miles per hour.	Locomotive:						Cars.
	10 221.	10 254.	10 302.	10 308.	10 301.	8 675.†	
Straight track:							
25	2	— 3	3	..	....	5	..
40	6	1	8	4	— 1	11	13
60	..	17	24	13	9	..	..
6° Curve:							
30	4	— 1	3	4	— 1	..	..
40	14	5	6	..	....	..	14
50	..	10	10	11	9	..	..
10° Curve:							
25	3	8	3	2	6	9	17
40	10	18	13	13	17	15*	26*

* At 35 miles per hour.

† The values do not include effect of counterbalance of locomotive drivers. The fourth driver showed on straight track an increase in stress due to counterbalance of 35% for the speed of 40 miles per hour, making with the 11% increase due to speed a total increase of 46 per cent.

The increase in stress with speed found with the Mikado type locomotive was smaller than that observed in previous tests. The speed effect found on straight track with the loaded cars was smaller than expected—14% increase in stress from 5 to 40 miles per hour.

Table 5 also gives for the curved track the average increase in stresses in rail with change in speed, averages for all the wheels of the locomotives or cars being used. As in curved track, centrifugal force, superelevation of outer rail, turning forces, and restraint of truck frames cause stresses to vary in the inner and outer rails under the several wheels at the different speeds, the values of the ratios given in Table 5 were based on the average of the mean stresses in the two rails under all the wheels of the electric locomotives, under the drivers of the Mikado type locomotive, and under

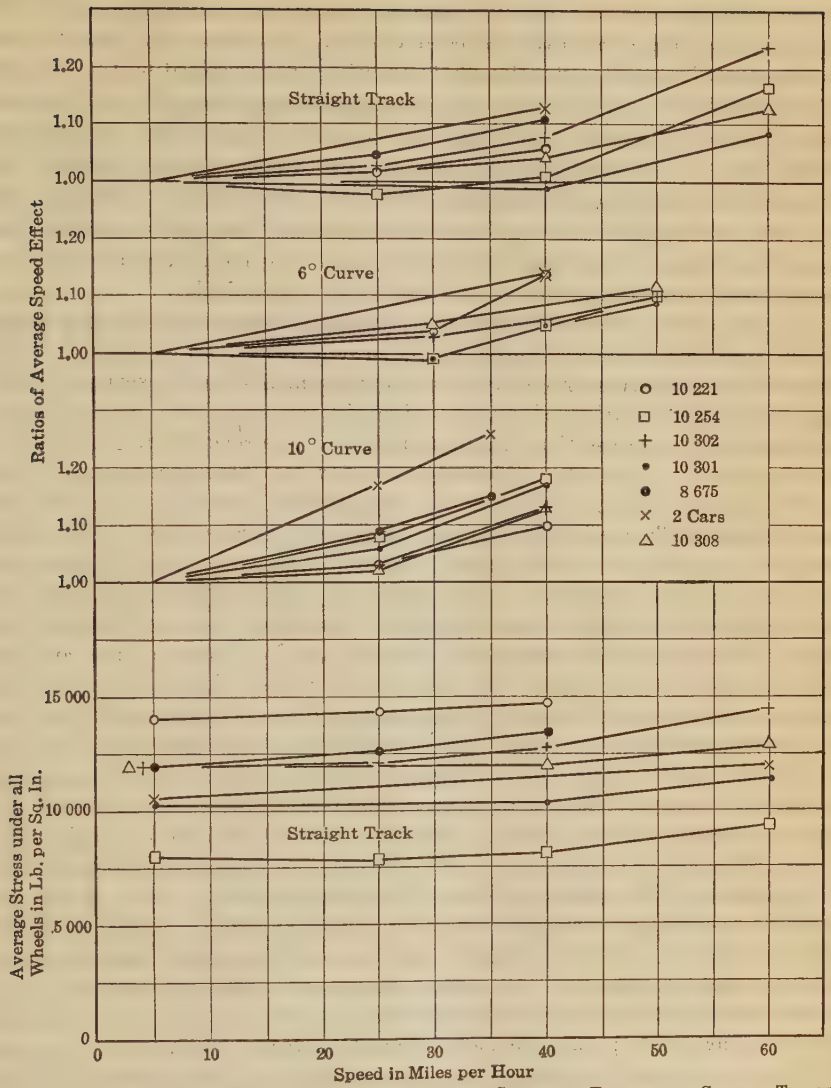


FIG. 14.—RATE OF INCREASE IN STRESS IN RAIL ON STRAIGHT TRACK AND CURVED TRACK IN TERMS OF THE STRESS AT 5 MILES PER HOUR; ALSO VALUES OF STRESS ON STRAIGHT TRACK.

all the wheels of the loaded freight cars. This method is not an entirely rational one, but it may be expected to give a fairly good way of judging of the effect of speed on curved track. The general trend of the effect of speed with the electric locomotives on the 6° curve and the 10° curve may be seen from Fig. 14; for the 10° curve the ratios for the Mikado type locomotive and for the cars have also been plotted. It is seen that the effect of speed on the 6° curve for the electric locomotives, as measured by the increase in average stress in rail, is not particularly different from that on straight track. On the 10° curve at 40 miles per hour the speed effect is more than double that on the straight track and the 6° curve, though even on the 10° curve the speed effect is relatively small. The tests with the loaded freight cars made on the 10° curve at 40 miles per hour show twice as great an increase in stress in rail over that at 5 miles per hour as was found on straight track and the 6° curve, 14 and 26%, respectively.

The data taken with the Mikado type locomotive were utilized to determine the effect of the counterbalance of this locomotive on stress in rail. The values of the stresses recorded by the eight instruments for the several speeds, when plotted for position of counterweight in the revolution of the driver, were concordant, more so than has been usual in those reported in the previous progress reports. The belt of plotted points for the three speeds (not reproduced here) was narrow except for the main driver, generally extending not more than 3 000 lb. per sq. in. on each side of an average line. At 25 miles per hour, the effect of counterbalance was not sufficiently marked to permit curves to be drawn expressive of this effect. The plots for 40 miles per hour clearly defined the effect of counterbalance throughout the revolution of the driver. Table 4 gives the stresses at the high and low points of the counterweight for 40 miles per hour and the average stresses for the other speeds, together with the rate of increase of stress over that at 5 miles per hour due to counterbalance and to speed. It is seen that at 40 miles per hour the main driver has an underbalance effect, with an increase in stress in rail due to counterbalance of 16% of the stress found at 5 miles per hour and 8% more stress due to speed change. The plots showed that for this driver the highest stress in the rail occurred after the pin had passed the low point of the revolution. The greatest effect of counterbalance is found under the fourth driver, 35%, making with the 11% effect of speed a combined speed and counterbalance effect at 40 miles per hour of 46%, and giving an average stress at the low point of the counterweight of 20 500 lb. per sq. in., a greater average stress than was found under the other drivers of this locomotive. The first driver had the next greatest increase, 30% due to counterbalance and 9% due to speed. The values reported are the averages of those found in the two rails and for the eight instruments used. Except perhaps for the fourth driver the counterbalance effect of this locomotive compares favorably with that of most of the Mikado type locomotives heretofore used, and the speed effect is markedly low for this type of locomotive.

9.—Lateral Bending Stresses and Lateral Movement of Rail on Straight Track.—As noted in the previous progress reports, the rail of straight track

bends laterally, sometimes outwardly and sometimes inwardly, as the wheels pass over it. The differences in the stresses in the two edges of the base of rail may be taken to measure the intensity of the lateral flexure, though it does not give the amount of the lateral movement or even its direction. The ratio of the stress at the outside edge of the base of rail to the mean of the stresses at the two edges is a convenient ratio to use in making comparisons of the lateral bending effect at different points along the track or on different runs or under different wheels, if the load of the wheel or the mean stress under the wheel do not differ greatly. If the ratio is greater than 1.00, the rail has been bent outwardly of the track; if less than 1.00, the bending is inwardly. Unless there is considerable tilting of the rail under the load in an unusual way, outward bending of the head of the rail goes with outward bending at the base, a dissimilar movement would imply very great twisting action. It will be well to keep in mind that a ratio of stress at the outside edge to mean stress having a value of 1.20 means that the stress at the outside edge is 1.50 times that at the inside edge, one of 1.33, that it is double, and one of 1.50, that it is treble. As the amount, direction, and conditions of action in the lateral bending of the rail may throw light on the action of the track and its maintenance, it will be well to study the results of the tests.

It has been found that the lateral bending of the rail differs with different wheels and different runs and also that it may vary at different points along the track.

Tables 6 and 7 give for straight track, for the different speeds, ratios of stress at the outside edge of base of rail to mean stress for the several locomotives and for the loaded freight cars. The values are the averages for each rail for all the runs; they thus represent the average results for the four points on each rail at which the instruments were placed. A study of Tables 6 and 7 shows that the average ratios are both greater and less than 1.00, that those less than 1.00 rarely go below 0.90, and that although most of the averages for the several speeds are less than 1.10, yet a number are greater than 1.20, and one wheel at one speed averages above 1.30. It is seen that the average for the left rail is less than that for the right, but the difference in general is not great. Most locomotives have fairly even values for most of the wheels, as note Locomotives 10254 and 10308. Locomotive 10302 has two drivers on the left side and three drivers and a trailer on the other giving values above 1.10. With the Mikado type locomotive, three drivers and two tender wheels range above 1.10. Three wheels of the eight on the loaded cars show the higher ratios. An examination has been made of the contours of the tires of the locomotives, but no definite correlation was found between the condition of the tires and observed stress ratios.

As a relatively few applications of the lateral pressures that give the extreme values (high or low) may be expected to affect track maintenance more seriously than many applications near the average value of the ratio, it will be well to consider the range and distribution of individual values. Fig. 15 shows individual observations for the Mikado type locomotive, the

TABLE 6.—Values of the Average Ratio of the Stress at the Outside Edge to Mean Stress in Base of Rail of Straight Track for the Locomotives of the Chicago, Milwaukee and St. Paul Railway.

Locomotive.	Speed, in miles per hour.	Rail.	Rear End Truck.		Driver.								Head End Truck.	
			2.	1.	8.	7.	6.	5.	4.	3.	2.	1.	2.	1.
10221	5	Left	0.94	1.05	0.97	1.06	1.04	0.98	1.00	1.01	1.01	1.01	0.94	1.13
		Right	1.05	1.04	1.05	1.04	1.07	1.05	1.03	1.05	1.02	1.02	1.03	1.16
	25	Left	0.99	1.07	0.97	1.01	1.04	0.96	0.99	0.99	1.03	0.97	1.04	1.12
		Right	1.02	1.05	1.05	1.04	1.10	1.06	1.05	1.06	1.05	1.03	1.06	1.11
	40	Left	0.96	1.14	0.97	1.05	1.13	1.00	1.03	1.04	1.05	1.02	0.94	1.14
		Right	1.01	1.04	1.04	1.04	1.16	1.05	1.01	1.04	1.02	0.99	1.00	1.08

Locomotive.	Speed, in miles per hour.	Rail.	Rear End Truck.		Driver.												Head End Truck.	
			2.	1.	12.	11.	10.	9.	8.	7.	6.	5.	4.	3.	2.	1.	2.	1.
10254	5	Left	....	0.94	0.96	0.93	1.01	0.92	0.94	0.95	0.93	1.01	0.98	1.13	0.97	1.02	....	0.99
		Right	....	0.96	0.93	1.01	1.03	0.92	0.99	1.03	1.01	0.93	0.89	1.16	0.90	0.98	....	1.01
	25	Left	....	0.98	0.96	0.98	0.99	0.94	0.95	0.95	1.10	1.05	1.00	1.17	0.98	1.11	....	0.98
		Right	....	0.95	0.96	1.01	1.04	0.99	1.08	1.10	1.07	1.00	1.01	1.23	0.96	1.05	....	1.03
	40	Left	....	1.06	1.11	1.03	1.11	1.02	1.11	1.01	1.05	1.14	1.15	1.23	1.03	1.13	....	0.99
		Right	....	1.02	1.11	1.07	1.05	1.05	1.05	0.98	1.01	1.06	0.95	1.26	0.99	1.07	....	1.02
	60	Left	....	1.06	1.01	0.95	1.07	1.10	1.11	1.15	1.06	1.22	1.23	1.25	1.11	1.17	....	1.08
		Right	....	1.01	0.96	1.02	1.06	1.09	1.06	1.26	1.06	1.07	1.02	1.18	0.99	0.95	....	1.03

Locomotive.	Speed, in miles per hour.	Rail.	Rear End Truck.		Tender.				Trailer.	Driver.				Head End Truck.	
			2.	1.	4.	3.	2.	1.	1.	4.	3.	2.	1.	2.	1.
8675	5	Left	....	....	0.95	1.04	0.99	1.16	0.84	1.06	0.98	1.08	1.12	....	1.04
		Right	....	....	0.95	0.97	0.91	1.17	1.07	1.15	1.08	1.15	1.12	....	0.99
	25	Left	....	....	0.92	1.01	1.02	1.16	0.82	1.07	0.92	1.03	1.09	....	0.98
		Right	....	....	0.95	1.02	0.91	1.19	1.04	1.18	1.15	1.17	1.04	....	0.99
	40	Left	....	....	0.93	1.03	1.05	1.14	0.84	1.04	0.94	1.08	1.12	....	1.02
		Right	....	....	0.93	0.97	0.92	1.10	1.03	1.15	1.05	1.16	1.02	....	1.01

TABLE 7.—VALUES OF THE AVERAGE RATIO OF THE STRESS AT THE OUTSIDE EDGE TO MEAN STRESS IN BASE OF RAIL OF STRAIGHT TRACK FOR THE LOCOMOTIVES AND CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

Locomotive.	Speed, in miles per hour.	Rail.	Rear End Truck.		Driver.			Trailer.				Driver.			Head End Truck.	
			2.	1.	6.	5.	4.	4.	3.	2.	1.	3.	2.	1.	2.	1.
10 302	5	Left	0.88	0.99	0.90	0.98	1.19	....	....	0.85	1.01	1.10	1.13	0.98	1.06	1.03
		Right	0.97	0.87	1.00	1.01	1.06	....	....	1.08	1.16	1.22	1.22	1.33	0.73	1.01
	25	Left	0.93	0.95	0.90	0.92	1.06	....	....	0.80	0.95	1.05	1.13	0.95	1.03	0.99
		Right	0.98	0.82	0.97	1.00	1.11	....	....	1.04	1.11	1.23	1.23	1.28	0.69	1.00
	40	Left	0.92	0.94	0.94	0.93	1.09	....	....	0.89	1.02	1.09	1.15	0.97	1.09	1.04
		Right	1.05	1.00	1.01	0.99	1.10	....	....	1.03	1.11	1.21	1.24	1.27	0.76	1.02
	60	Left	1.02	1.00	0.97	0.99	1.10	....	....	0.89	1.06	1.05	1.13	1.05	1.08	1.00
		Right	1.04	1.00	1.01	1.01	1.06	....	....	1.03	1.05	1.12	1.19	1.21	0.99	1.07
10 308	5	Left	0.97	1.05	1.09	1.01	1.05	....	....	1.01	1.02	1.03	0.97	0.96	1.06	1.05
		Right	1.06	0.97	1.15	1.09	1.10	....	....	1.02	1.07	1.05	0.97	1.01	0.93	1.00
	40	Left	0.98	0.92	1.16	1.08	1.01	....	....	1.05	1.03	1.07	0.98	1.02	1.06	1.08
		Right	1.01	1.09	1.24	1.10	1.08		....	1.02	1.07	1.06	1.01	1.01	1.14	1.05
	60	Left	0.98	0.95	1.18	1.06	1.08	....	....	1.03	1.05	1.07	1.04	1.07	1.04	1.02
		Right	1.00	1.08	1.21	1.05	0.97	....	....	1.02	1.03	1.05	0.93	1.04	1.13	1.06
10 301	5	Left	0.98	0.95	0.97	0.94	0.96	1.04	1.02	1.07	1.06	0.99	0.99	1.01	1.20	0.92
		Right	1.05	1.10	1.04	0.99	1.05	1.20	1.09	1.08	0.96	1.07	1.04	1.06	1.03	1.21
	40	Left	0.95	0.93	1.00	0.93	0.94	1.05	0.94	0.98	0.97	0.95	0.97	0.98	1.13	0.99
		Right	1.12	1.21	1.06	0.97	1.00	1.14	1.09	1.12	1.09	1.05	1.04	1.01	0.94	1.21
	60	Left	0.99	1.27	1.10	1.04	1.08	1.11	1.08	0.92	1.15	1.07	1.02	1.14	1.37	1.05
		Right	0.94	1.17	0.97	0.93	0.96	1.10	1.06	1.06	1.03	1.01	1.00	0.97	0.97	1.18

			Second Car.				First Car.			
			4.	3.	2.	1.	4.	3.	2.	1.
Loaded cars	5	Left	0.94	1.08	1.00	1.00	0.95	1.06	1.05	1.08
		Right	0.99	1.13	0.94	0.99	0.91	1.10	0.95	1.23
	40	Left	0.99	1.07	1.06	1.08	1.06	1.08	1.11	1.04
		Right	1.03	1.11	1.02	1.02	1.05	1.18	1.10	1.06

values for the several instruments being distinguished by symbols and each instrument being assigned a part of the space given for a wheel and in the order of the instruments along the track. It is seen that for the Mikado type locomotive the range is from less than 0.60 to more than 1.40. It is also seen that although the average ratio on the two rails for all the drivers and the trailer is 1.07, there are a large number of individual values between

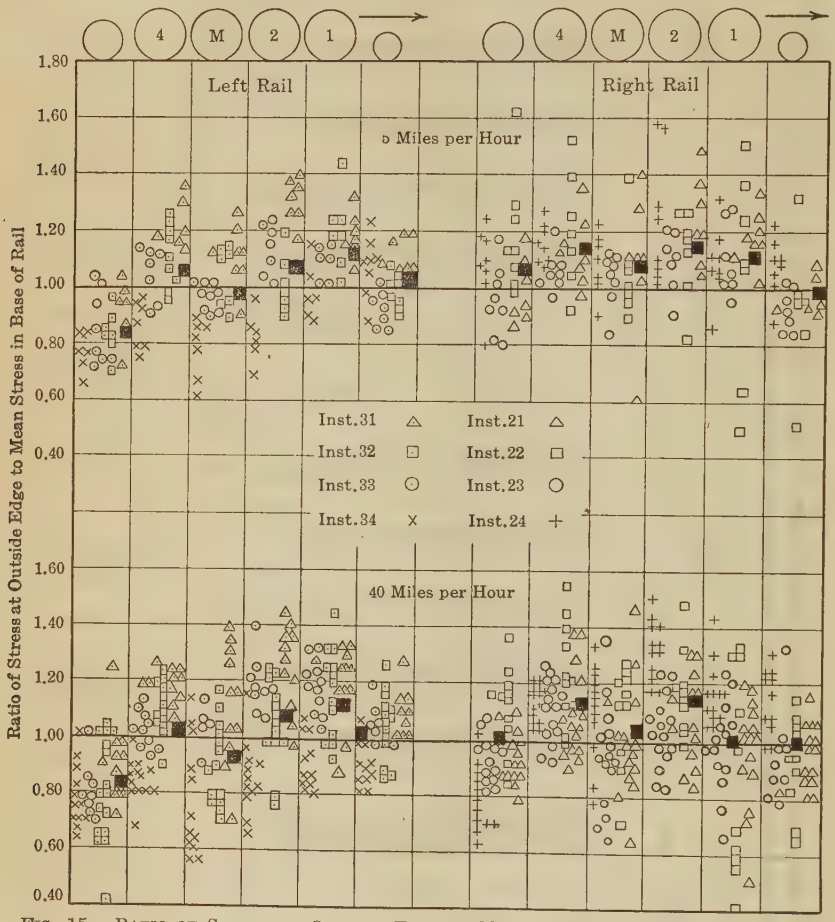


FIG. 15.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 8675 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

1.20 and 1.50, and relatively few less than 0.80. It is apparent that many relatively large outward pushes on the rail were given in the passage of the drivers over this track and that the effect of the outward bending of the rail upon upkeep would be larger than the indication given by the average of all the ratios. It is noticeable that at a particular instrument one wheel would give a relatively high average value and another wheel a lower one, and at another instrument the reverse was true. In general, the range of values

recorded by an instrument was considerable, the amount of the range being much the same for all the instruments, but, in a few cases, at a single wheel the values were quite compactly grouped.

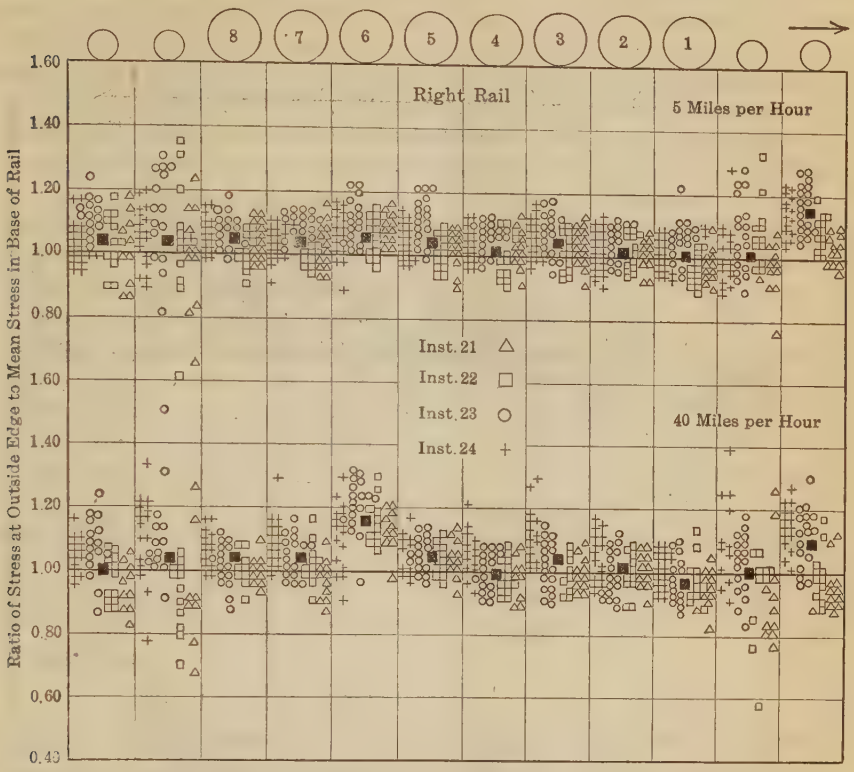


FIG. 16.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10221 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

Space will not permit reproducing all the diagrams of the same kind for the other locomotives. They varied greatly. In a few cases the ratios were found to be quite uniform at the various instruments and for the several wheels; thus, in Fig. 16, the values of the ratios on the right rail with Locomotive 10221 at 5 miles per hour keep within a range of 0.08 of the average ratio and are very compactly grouped. More generally, however, the values of the ratios are widely distributed and may vary for different wheels; thus, Locomotive 10302 at 40 miles per hour (see Fig. 17) gives ratios of stresses in the right rail that vary from 0.40 to 1.55, this range occurring under adjoining wheels. It is evident that the variations in values (high and low) at the several instruments on one rail differed considerably from those at the several instruments on the other rail. The average value for the two rails also differed. In some cases, as for one instrument near a rail joint, the values were in the high part of the range (or the low part) for all the locomotives, this being the first instance in the test work that the

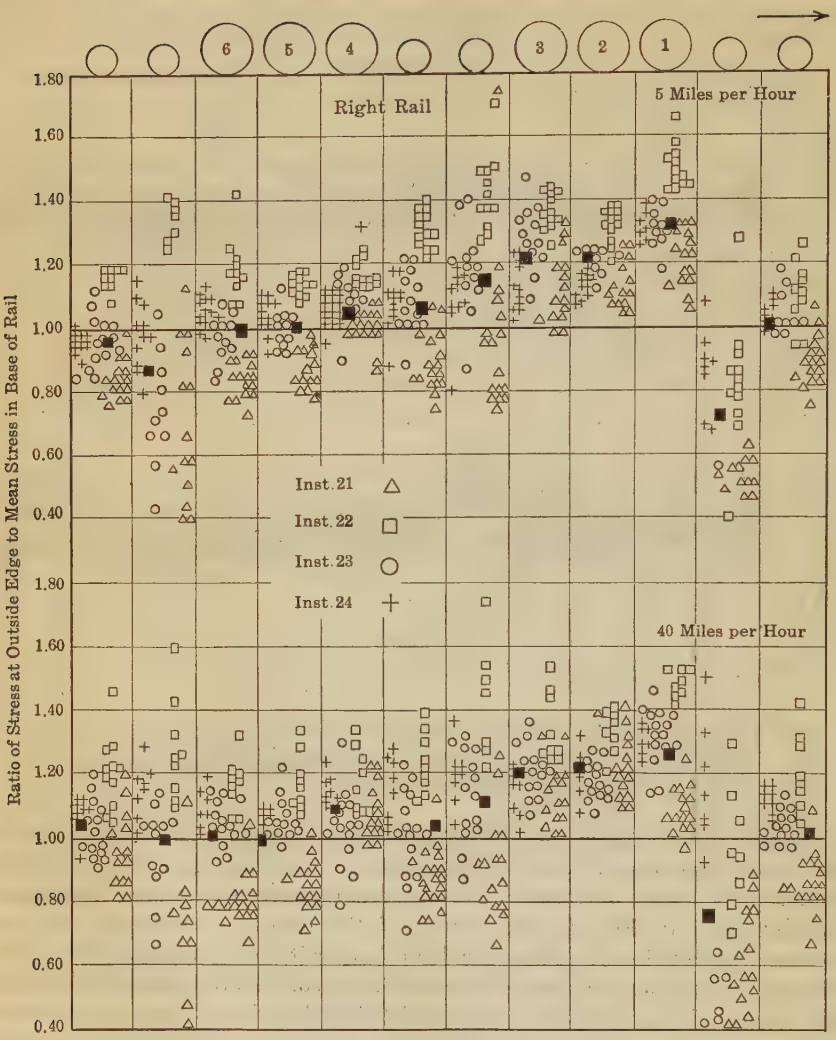


FIG. 17.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. LOCOMOTIVE 10302 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

ratios for a given instrument near a joint were consistently higher or lower than those at the other side of the joint. Generally, however, a given wheel develops at a given instrument little variation in ratios, the range from an average usually being not more than 5 per cent. Some wheels differ markedly from most of the others of the locomotives; thus, the third right driver of Locomotive 10254 for all speeds generally gave ratios at the four instruments between 1.10 and 1.60, the average being 1.21, while for the remaining wheels of the right side the average ratio was slightly more than 1.00 and the individual observations are fairly uniformly distributed through the usual range; on the left rail the range in ratios is still greater and for certain wheels there are marked differences in the values at different instruments. It must be borne in mind that the stresses developed by Locomotive 10254 are small as compared with the other locomotives and that given variations in stress will show a higher range in the ratios.

For the loaded freight cars (for which fewer observations are available than for the locomotives) the values of the ratio of stress at outside edge to mean stress in base of rail, in general, have a much smaller range and are more compactly grouped and vary less from instrument to instrument along the track. At 5 miles per hour, only one wheel of the first car gave an average value of the ratio of more than 1.20 and, at 40 miles per hour, the higher values are relatively few in number. In the second car only one wheel gave a ratio greater than 1.10. Strangely, at 5 miles per hour, most of the high individual ratios were recorded under one wheel by one instrument and at 40 miles per hour under another wheel by another instrument. The lateral pressure on the rail given by these cars may be said to be not excessive.

As the section modulus of the rail, $\frac{I}{c}$, about a vertical axis is only about one-fifth of that about a horizontal axis, it is apparent that the bending moment corresponding to a given lateral bending stress is only one-fifth of the vertical bending moment which would give an equal vertical bending stress. The value of the observed lateral bending stress may be found by multiplying a mean stress in base of rail by a factor obtained by subtracting 1.00 from the ratio of stress at outside edge to mean stress corresponding to the given observation. Thus, for the right first driver of Locomotive 10302, by using the ratio, 1.33, given in Table 7 and the corresponding mean stress in base of rail given in Table 3, 13 500 lb. per sq. in., the lateral bending stress is seen to be 4 500 lb. per sq. in. An individual observation giving a ratio of 1.38 with a mean stress of 22 000 lb. per sq. in. corresponds to a lateral bending stress of 8 400 lb. per sq. in.

It is evident from a study of the tests that the lateral bending of the rail is caused both by the condition of the rail and the rail support (tie and underlying ballast) and by the action of the locomotive and cars. At two of the eight instruments, there was marked outward bending of the rail for all the locomotives, as measured by the average stresses for all wheels, and at one instrument, marked inward bending. At all the other points of

attachment of instruments, the several locomotives gave different effects. That is, at three of the eight points the support of the rail was such as to favor the lateral bending of the rail one way or the other. At the other five points the locomotive and cars were evidently the governing cause of the lateral bending. It is apparent, of course, that the variability in the lateral bending shown by the considerable range in bending stresses and ratios is due to variations in the action of the locomotive in passing over the track on different runs.

Two trial runs at slow speed with Locomotive 10302 gave maximum outward lateral movements of the head of one rail of 0.03 and 0.05 in. when the wheels were passing over the point of observation. It is seen that the lateral movement is slight in this case and that nothing of value was learned from the few measurements made.

To learn something of the position of the flanges and treads of the wheels with respect to the head of the rail a few tests on straight track were made near the end of the work by taking impressions on copper wire as the locomotive passed over the test section. As the method of using a punch-mark on the head of the rail had not then been developed, the position of the wheel bearing on the rail is uncertain except for the wheels having flange contact with the side of the head. It was found for Locomotive 8675 that certain wheels ran close to the gauge side of the rail at all speeds. Some wheels showed a broad contact with the rail, others a more concentrated contact. The flange of the first driver on the right side and usually also that of the fourth driver, ran close to the rail. The flange of the trailer and first wheel of the tender on the left side also ran close to the rail. In the runs at 40 miles per hour with Locomotive 10308, the flanges of the drivers on the right side of the second unit ran close to the rail. Under the other wheels the flanges were away from the rail and the pressures were concentrated near the center of the head of the rail or somewhat inside it. At 40 miles per hour, the flange of the first wheel on the left side ran close to the rail on one run and that on the other side on the other run. The flanges of other wheels also ran close to the rail. These observations are described at this length in order to bring out the variability of the positions and directions of pressures of wheels on rail and thus to point to some of the variable conditions that result in great variations in the amount and character of both vertical and lateral bending stresses in the rail.

10.—The Action of Curved Track.—In the third progress report, under the heading, "The Action of Curved Track",* the way in which the passage of a locomotive and cars around curved track may be expected to affect the track through transfer of load from rail to rail and wheel to wheel and the way in which lateral pressures on the rail may be produced, were discussed. Centrifugal force, transverse inclination of track, flange and tread lateral pressures, spreading of rails, and other matters were considered. In particular, it was brought out that the lateral force of the flange of the leading truck wheel or first driver or both against the outer rail necessary to change

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 973.

the direction of all the drivers grouped in a single frame in traversing a curve brought into action a lateral pressure against the inner rail by the wheels at or near the center of rotation about which the turning of the locomotive takes place and likewise an outward pressure on the outer rail by the rear outside driver. It was further shown that the vertical load transferred to the rail by the inside driver nearest the center of rotation might be much greater than its normal vertical load. The lateral forces thus set up and the increase in the load on one or more of the drivers which may be developed, greatly increase the lateral and vertical moments developed in the rail and therefore have important effects on track maintenance as well as on the rail itself. For details of the discussion reference is made to the third progress report, but a résumé of some of the statements and conclusions follows:

(*a*).—The longitudinal slip along a rail of the inside or outside wheel of a pair on the same axle made necessary by the difference in length of the two rails of curved track is considered to affect the lateral forces acting on the rails only as it may modify the coefficient of friction and thus also the magnitude of the force required to produce lateral slipping of the wheel on the rail. The effect of the coning of the wheels is considered to be negligible as an element in the problem.

(*b*).—The centrifugal force and the transverse inclination of the track necessarily affect the vertical and lateral pressures on the rails to an appreciable extent. The following approximate formulas were given for the pressure on the outer and the inner rail, respectively, to be expected from analytical considerations to correspond to the weight, W, applied to the rail through one wheel for straight and level track, when the speed is such that the centrifugal force is negligible:

$$R' = W\left(1 - 2\frac{h}{g}\frac{e}{g}\right) \quad \ldots\ldots (29)*$$

$$R'' = W\left(1 + 2\frac{h}{g}\frac{e}{g}\right) \quad \ldots\ldots (30)*$$

In these equations, g is the distance between the bearing points of the wheels on the two rails (taken in calculations as 59 in. for the curved track used in the tests), e is the superelevation of the outer rail, and h is the distance of the center of gravity of the locomotive or car from the level of the top of the rails. The equations were based on the assumption that the bodies of the locomotive and cars retain the same positions with respect to the axles of the wheels as are found on straight track. The tilting due to changed compression of the springs may modify the pressures somewhat.

Taking the load on a rail as equal to the algebraic sum of that due to Equation (29) or Equation (30) and that due to the centrifugal force, approximate formulas for the reactions of the outer and inner rails, respectively, were given as:

$$R_1 = W\left[\left(1 - 2\frac{e}{g}\frac{h}{g}\right) + \frac{2\,V^2 D}{85\,800}\left(\frac{h}{g} + \frac{1}{2}\frac{e}{g}\right)\right] \quad \ldots\ldots (31)\dagger$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 979.

† *Loc. cit.*, p. 980.

$$R_2 = W\left[\left(1 + 2\frac{e}{g}\frac{h}{g}\right) - \frac{2V^2D}{85\,800}\left(\frac{h}{g} - \frac{1}{2}\frac{e}{g}\right)\right] \ldots\ldots (32)^*$$

in which, V is the speed, in miles per hour, and D is the degree of curve. Likewise, the sum of the resulting lateral reaction or pressure against the outer rail of the curved track, F_1, and that against the inner rail, F_2, is obtained approximately by the equation:

$$F_1 + F_2 = \frac{2\,W\,V^2\,D}{85\,800} - 2\,W\frac{e}{g} \ldots\ldots (33)^*$$

the division of the lateral force between F_1 and F_2 being indeterminate.

(*c*).—In changing the direction of a group of drivers, as the locomotive traverses a curve, the outer rail presses against the flange of the outer front driver, or the outer wheels of the front truck, or both, according to conditions of speed, length of wheel-base, flexibility of frame and connections with other trucks, and other matters. This outer front driver and the inner driver on the same axle must be made to slip laterally on the rails inwardly of the curve. At some driver farther back, generally at a driver on the inner rail, there will be a center of rotation of the group of wheels connected to a single frame, about which the turning action of the frame takes place. The drivers ahead of the center of rotation, except the leading outer driver when it participates in the turning or directing action, will give pressures inwardly of the curve on both inner and outer rails. The drivers behind the center of rotation will swing outwardly of the curve and slip outwardly, and thus pressure in this direction will be developed against both the outer and the inner rail. In Fig. 18 the direction of the pressures of the wheels of a Mikado type locomotive against the rails is indicated. B is the center of rotation.

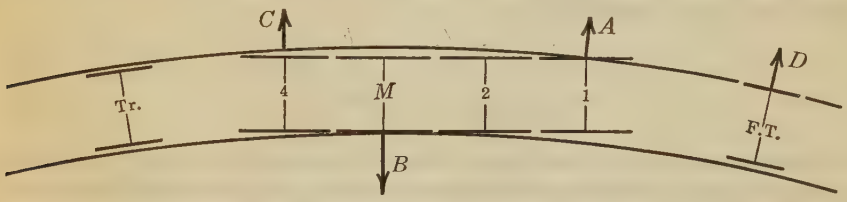


FIG. 18.—DIRECTION OF PRESSURE OF THE WHEELS OF A MIKADO TYPE LOCOMOTIVE ON THE RAILS OF CURVED TRACK.

The turning forces are at D and frequently at A also, flanges of the guiding wheels pressing against the head of the outer rail. It is evident that the lateral pressure of the wheel on the inner rail at the center of rotation, B, may constitute the principal force in opposition to the several lateral forces that are acting in the opposite direction. The pressure developed by the other drivers will be smaller in magnitude and will vary in direction according to conditions. The trailer may give pressures outwardly of the curve or inwardly according to the restraint, the coupling connection, and the degree of the curve. The deflections and lateral movement of the two rails as the locomotive moves around the curve, themselves takes a part in fixing the positions of the drivers and thus affect the development of the lateral reactions.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 980.

It is seen that the lateral forces tending to bend or to straighten the two rails, and thus to affect the whole track structure and its maintenance, are dependent on numerous factors.

In the four-wheel truck of a car, the turning is generally done by the flange of the outer forward wheel, and the rear wheels of the truck may be expected to approach a position radial to the curve.

11.—Results of Tests on Curved Track.—Space will permit the presentation of only a part of the data, but representative parts have been selected and general average values are given in the tables and diagrams.

Figs. 19 to 30, inclusive, give the stresses at the inside edge and the outside edge of the base of the outer and the inner rail under the wheels of six locomotives and two loaded freight cars at several speeds on the 6° curve and the 10° curve. The values given are the averages of the records of the four instruments on a rail for the runs made, representing generally about sixty observations for each value plotted. The variations of individual values from the average value reported were much the same as on straight track, but the differences between averages for the four instruments on a rail were greater than those for straight track, indicating that variations in the curvature of the rail and the conditions of its support affect the stresses developed to a greater degree than is the case with straight track.

In making comparisons between tests, reference may be made to the properties of the 90-lb. rail section given in Article 4, "The Track", and the weights of the locomotives and cars given in Figs. 1 and 2.

12.—Vertical Loads and Vertical Bending Stresses on Curved Track.—The mean stress in the base of rail (the average of the stresses observed at the two edges and generally called the vertical bending stress) is taken as representative of the bending of the rail in a vertical plane, or, more strictly, of the bending in a direction normal to the inclination of the track, the two effects being practically identical for the superelevation of track used in the tests, since the cosine of the angle of inclination is very close to unity. Although stresses in rail are not exactly proportional to the loads when the same total load is differently distributed among the several wheels or differently divided between the two rails, yet when the differences are not great the sum of the stresses in the rail under all the wheels will generally differ little on account of different divisions or distribution of load, and summations of the stresses for each rail may be useful in making comparisons and in checking up on the action of the locomotive and track. As has already been stated in Article 7, "Results of Tests on Straight Track", by means of the analytical method given in the first progress report and the solution of unknowns in equations based thereon, the load which will produce the given mean stress may be estimated quite approximately. It will be seen then that the values of the mean stresses in base of rail (vertical bending stresses) not only will permit comparisons with the stresses in straight track to be made, but will enable the vertical load to be distinguished from the lateral forces or loads that are also developed in passing around the curve. A close estimate may then be made of the distribution of vertical load among the wheels and between

the two rails as it is found on the various curves. A discussion will first be given based on the average stresses in the two rails, followed by one based on the loads at the individual wheels and the stresses in the rails.

As in the tests reported in the third progress report, the sum of the vertical bending stresses in the outer and inner rails under all the wheels of a locomotive for a given speed was found not to differ greatly from the sum of the corresponding stresses on straight track, being slightly larger for the curved track at the higher speeds. The total of the vertical load effects does not differ greatly in the two kinds of track.

The method used in the third progress report to compare the division of loads between the outer and the inner rail at the various speeds, as indicated by the stresses, with that given by Equations (31) and (32) of Article 10, "The Action of Curved Track", was applied to the tests. For this purpose the sum of the vertical bending stresses in the base of the inner rail under all the wheels of a locomotive was found and also the corresponding sum in the outer rail. As has already been stated, ratios of these sums to their mean may be expected to be fairly close to the ratios of the loads transmitted to the two rails in comparison with what would occur on straight track. In Table 8 are given the values so found, those determined from the loads calculated by the method already outlined, and also the ratios calculated by Equations (31) and (32). On the whole, the agreement between the two sets of ratios is very close. At the low speed the values derived for the stresses are nearer unity than the analytical values. Such matters as difference in the tilting of the rail for different speeds and differences in deflections of the springs and action of the equalizing system on the two sides of the locomotive are elements not considered in the derivation of Equations (31) and (32). It will be noted that the weights on the outer rail at the highest speeds used on the two curves are about 25% greater than that normally coming on one rail on straight track. Although these speeds, 50 miles per hour on the 6° curve and 40 miles per hour on the 10° curve, were one and two-thirds times the speed corresponding to the superelevation of the track, the locomotives ran the curves smoothly except where the alignment was not good. The effect of the low and the high speeds on the track structure is best judged from the lateral bending stresses in the rail, which will be discussed in Article 13. It will be noted that there is little difference in the ratios obtained direct from the stresses and those obtained through the load calculated from the stresses.

The vertical bending stress in base of rail (mean of the stresses in the two edges of the base and the average of all the observations with the four instruments in all the runs) under the various wheels of the locomotive and cars is recorded in Figs. 31 to 37 for both outer and inner rails for the speeds used in the tests on curved track. For convenience, the 6° curve is shown as curving to the right instead of to the left. Comparison may be made with the stresses under the same wheels in the tests on straight track, as given in Figs. 6 to 12 and in Tables 2, 3, and 4. In making a study of the vertical bending stresses it should be kept in mind that changes in the values of

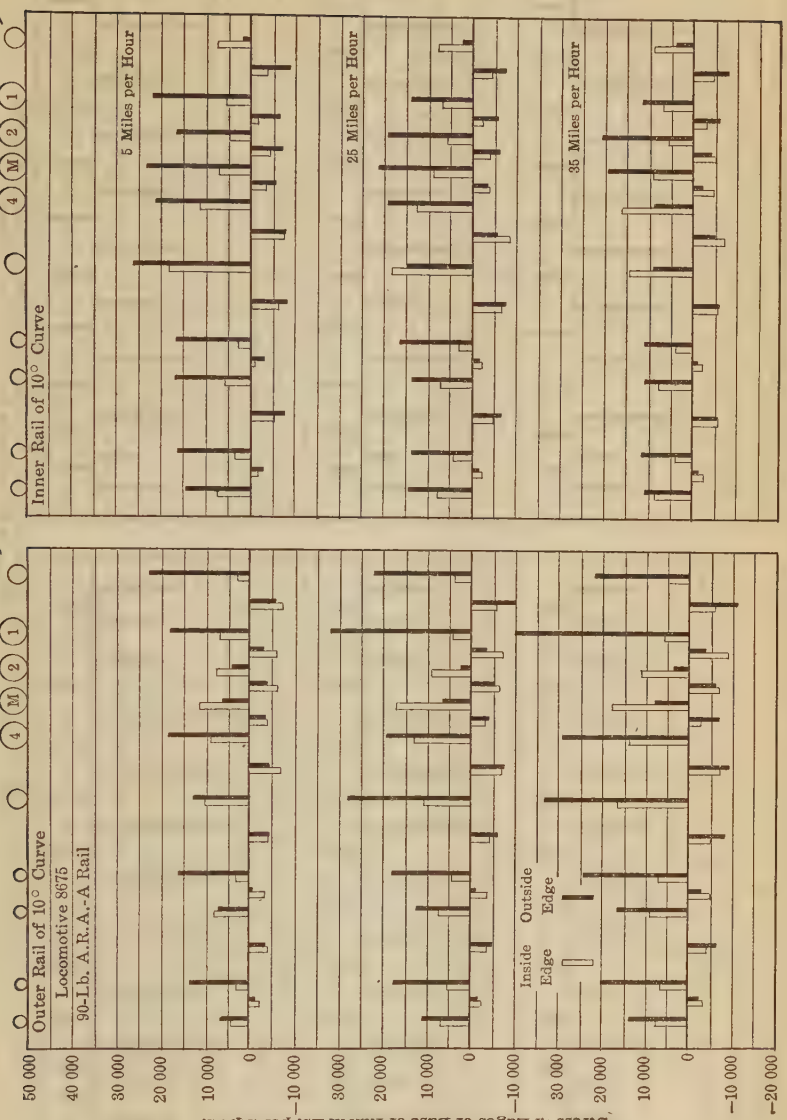


FIG. 19.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 8675 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

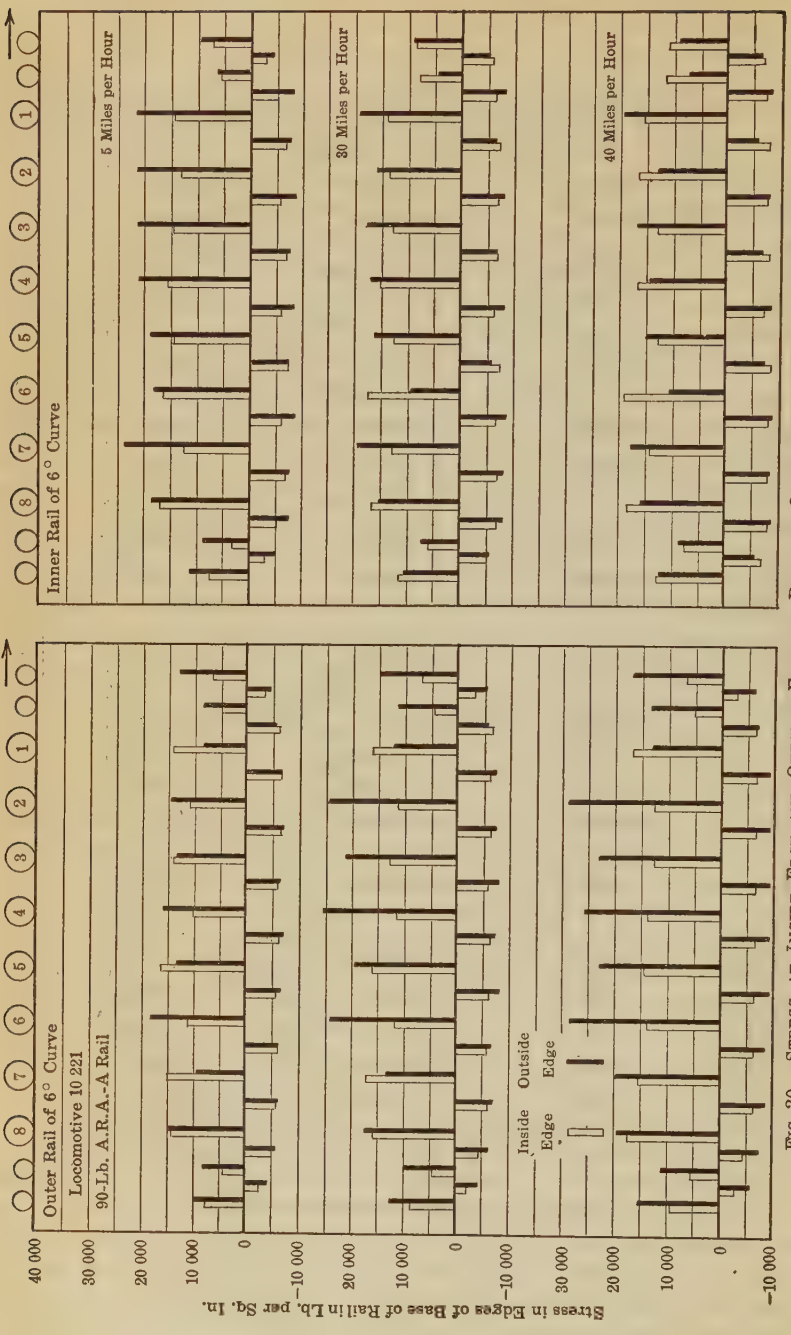


FIG. 20.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° CURVE. LOCOMOTIVE 10221 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

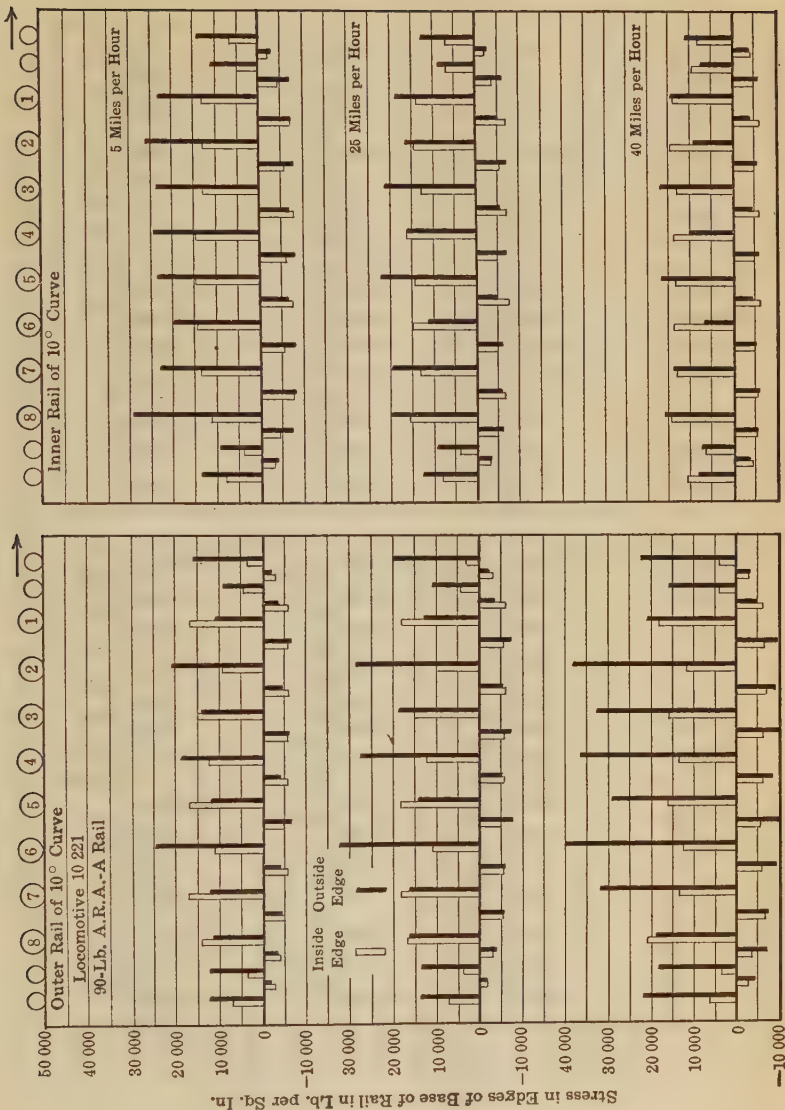


FIG. 21.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 10221 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

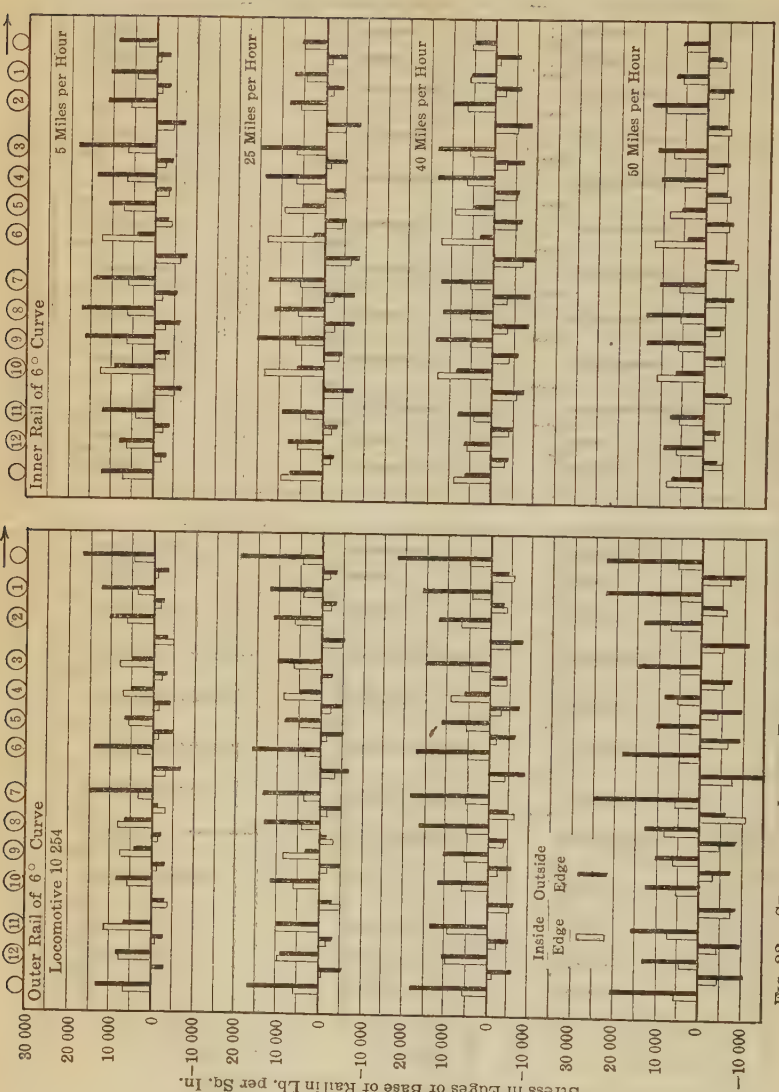

Outer Rail of 6° Curve
Locomotive 10 254
Inner Rail of 6° Curve
5 Miles per Hour
25 Miles per Hour
40 Miles per Hour
50 Miles per Hour
Inside Edge
Outside Edge
Stress in Edges of Base of Rail in Lb. per Sq. In.
30 000
20 000
10 000
0
−10 000
12 11 10 9 8 7 6 5 4 3 2 1

FIG. 22.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° CURVE. LOCOMOTIVE 10254 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

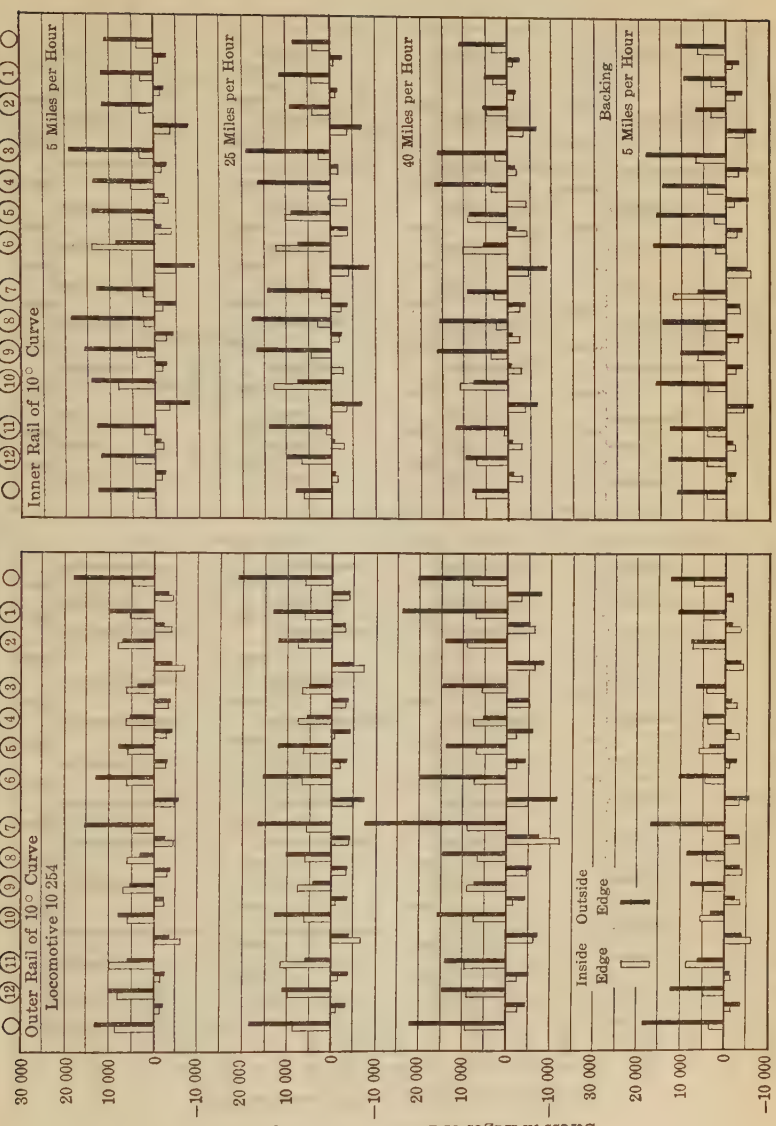


FIG. 23.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 10254 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

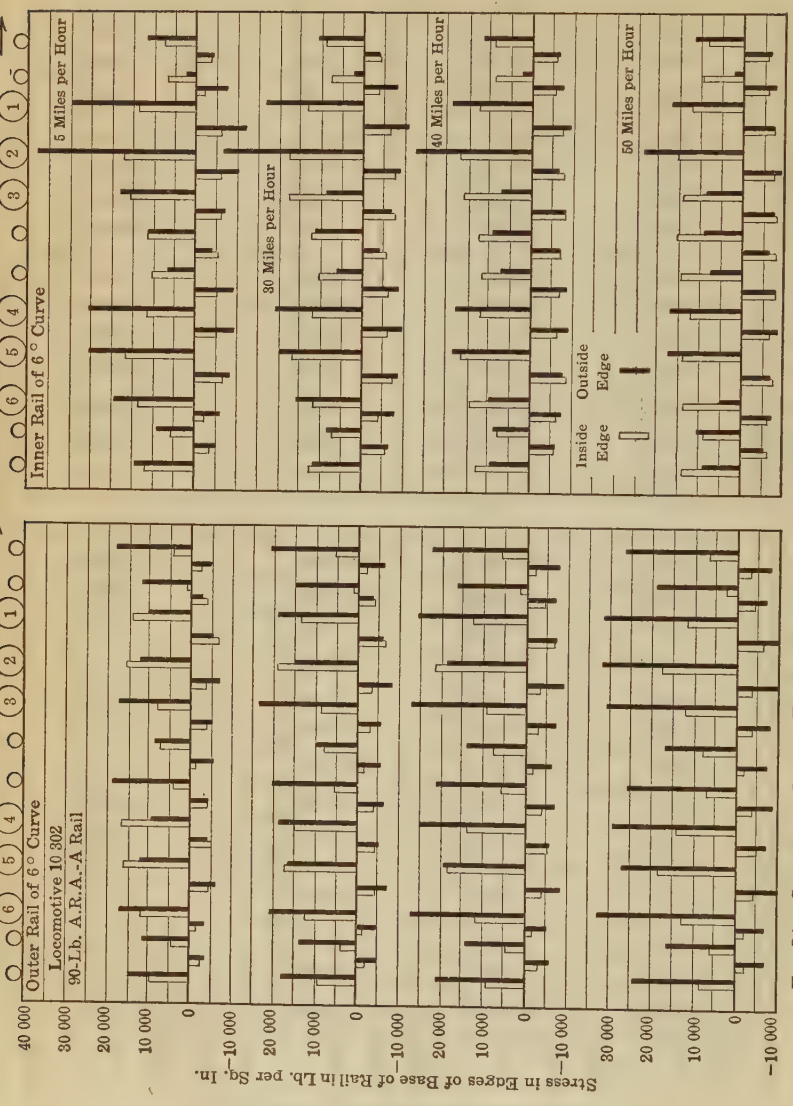


FIG. 24.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° CURVE. LOCOMOTIVE 10302 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

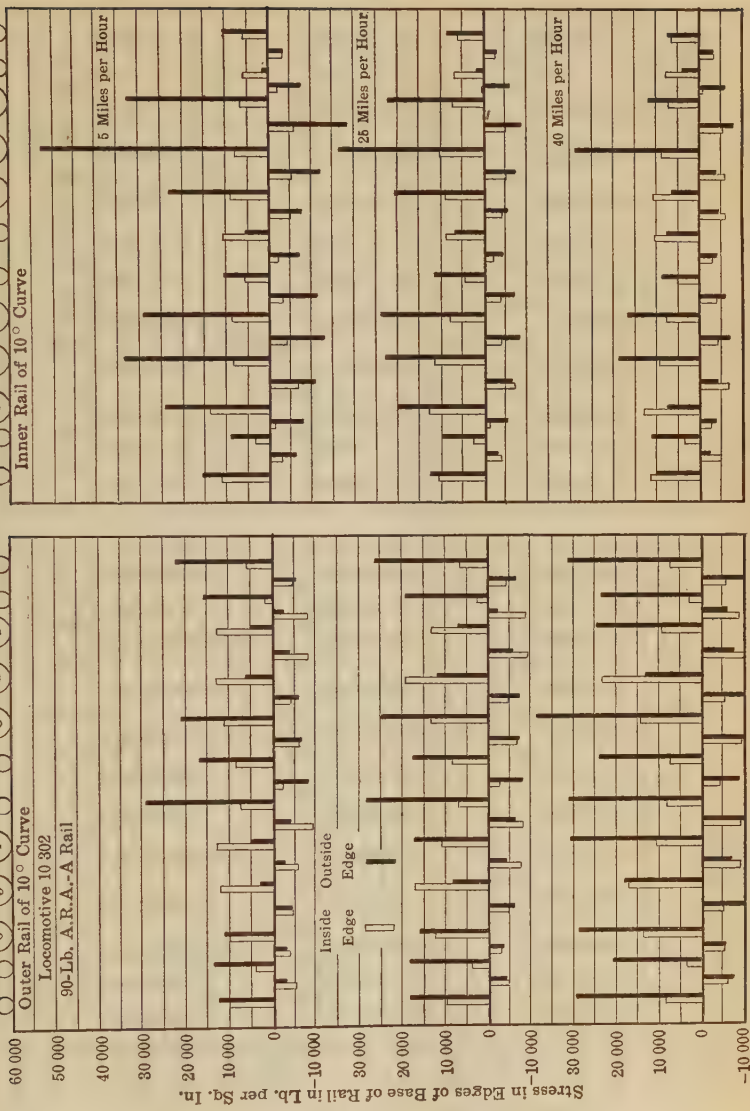


FIG. 25.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 10302 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

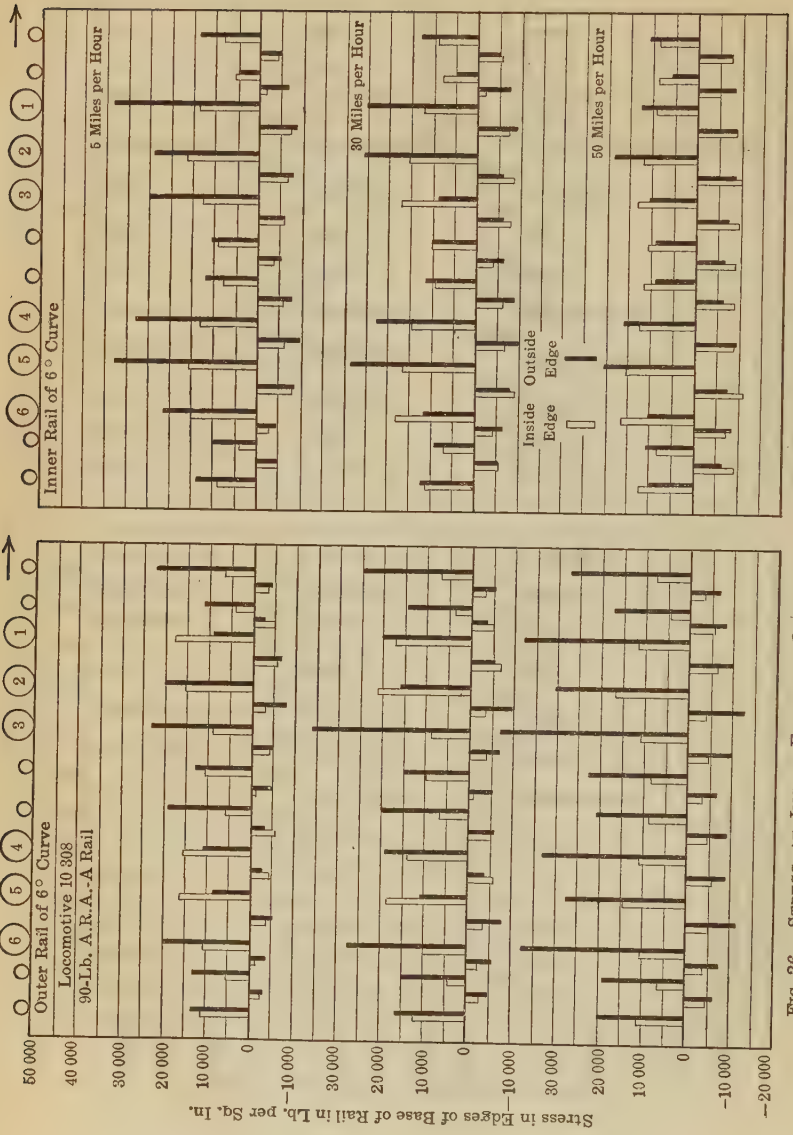


FIG. 26.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° CURVE. LOCOMOTIVE 10308 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

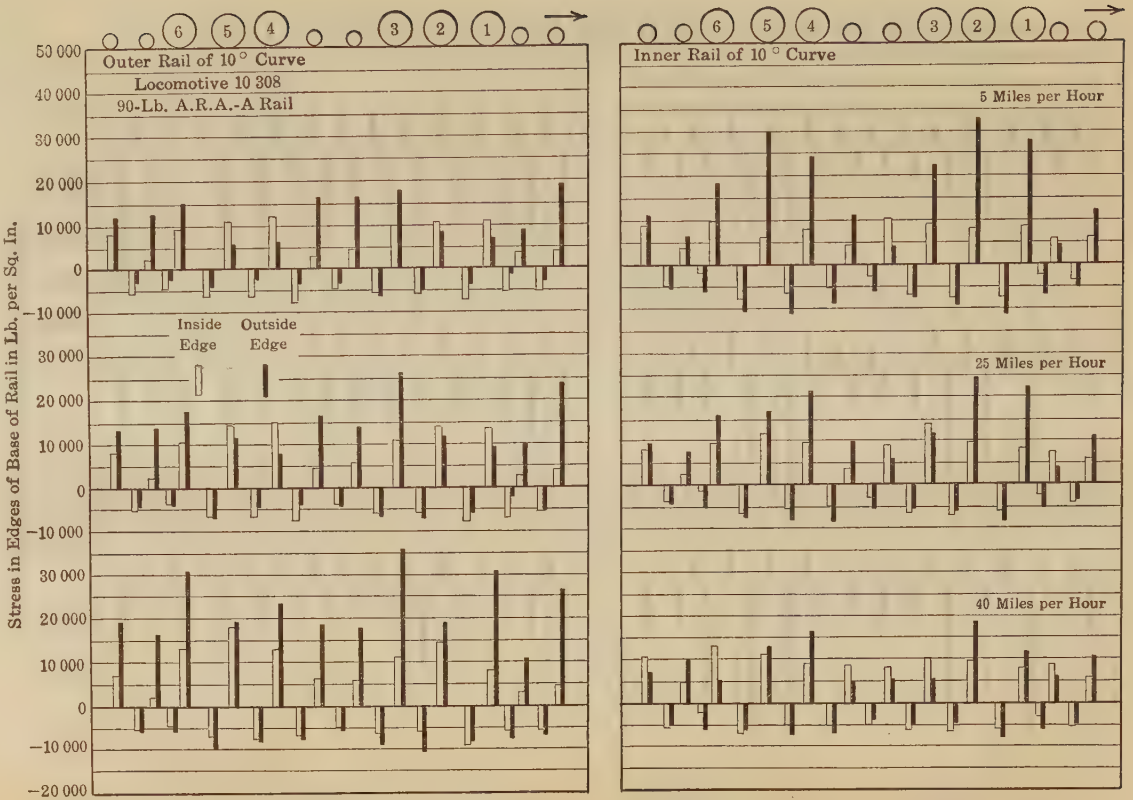


FIG. 27.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 10308 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

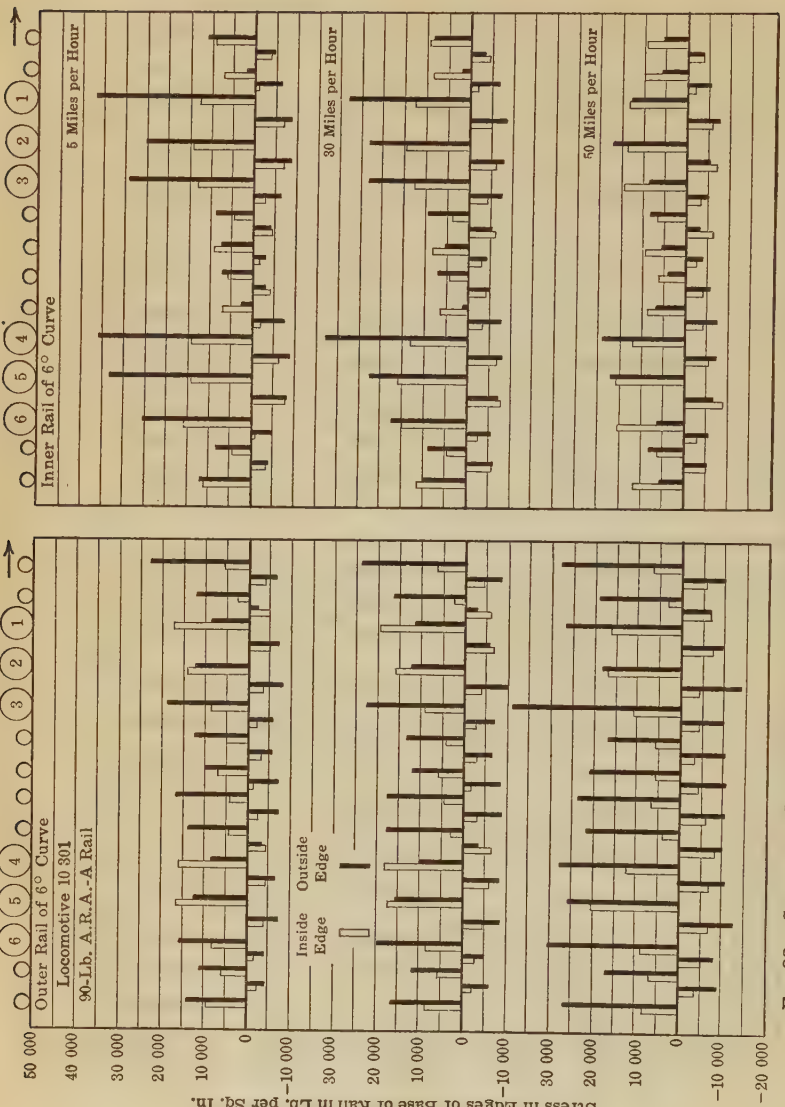


FIG. 28.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° CURVE. LOCOMOTIVE 10301 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

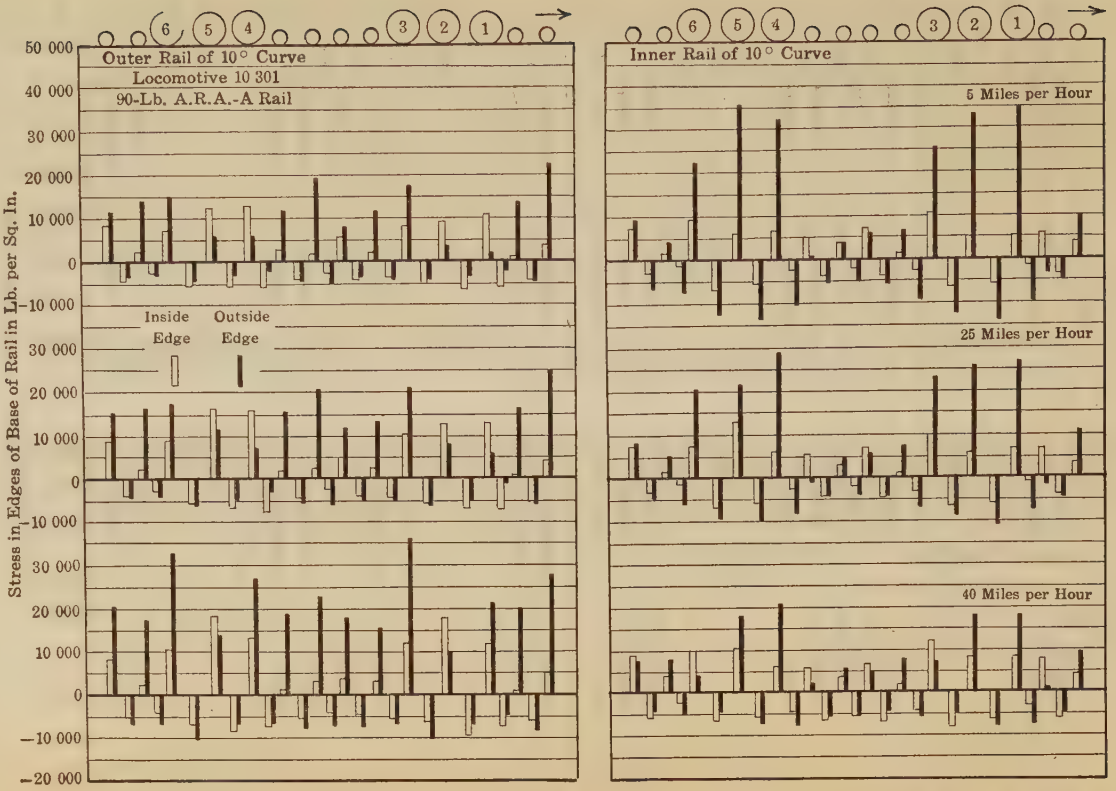


FIG. 29.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE. LOCOMOTIVE 10301 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

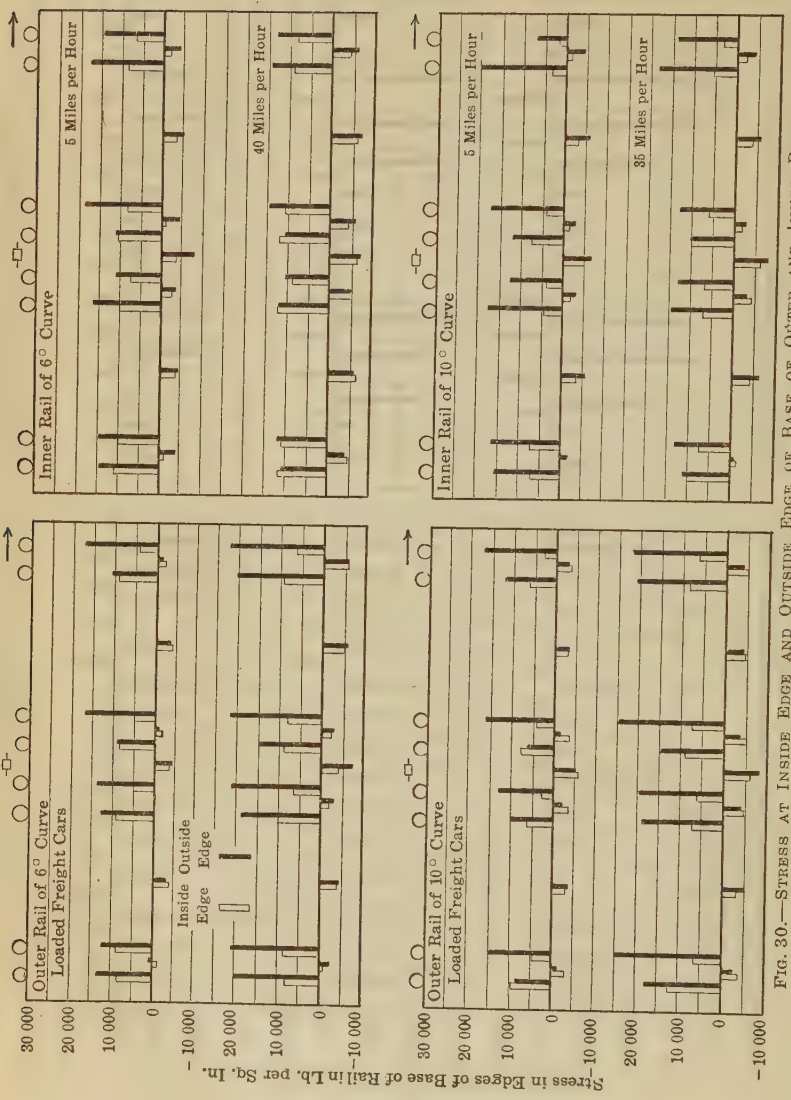


FIG. 30.—STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF OUTER AND INNER RAILS OF THE 6° AND 10° CURVES. LOADED FREIGHT CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

the vertical bending stresses at any wheel must be due principally to changes in the amount of load on the wheel either by itself or in combination with adjoining wheels.

The method already described was used for estimating the wheel loads necessary to produce the vertical bending stresses observed on curved track. For the sake of uniformity the results of the calculations for 5 miles per hour were changed proportionately to make the sum of the wheel loads equal the total weight of the locomotive. The factor used ordinarily differed from unity by 5% or less. For the other speeds the same factor was applied, so that the resulting load may be said to represent equivalent loads which would produce the stresses if applied statically or at a low speed as were found with the locomotive at the given higher speeds.

TABLE 8.—Ratio of Load on Inner and Outer Rail to Mean Load, Chicago, Milwaukee and St. Paul Railway.

Locomotive.	Degree of curve.	Super-elevation, in inches.	Speed, in miles per hour.	Analytical Values.		Values from Calculated Loads.		Ratios of Stresses.	
				Inner rail.	Outer rail.	Inner rail.	Outer rail.	Inner rail.	Outer rail.
10 221	6°	3.2	5	1.11	0.88	1.10	0.90	1.10	0.90
			30	0.99	1.02	0.97	1.03	0.96	1.04
			40	0.88	1.13	0.93	1.07	0.92	1.08
10 254	6°	3.2	5	1.10	0.90	1.09	0.91	1.09	0.92
			30	0.98	1.02	0.98	1.02	0.97	1.03
			40	0.89	1.12	0.89	1.11	0.87	1.13
			50	0.76	1.26	0.88	1.12	0.87	1.13
10 302	6°	3.2	5	1.13	0.87	1.13	0.87	1.13	0.87
			30	0.99	1.02	0.99	1.01	0.99	1.01
			40	0.87	1.14	0.90	1.10	0.90	1.10
			50	0.72	1.30	0.77	1.23	0.77	1.23
10 308	6°	3.2	5	1.13	0.87	1.07	0.93	1.07	0.93
			30	0.99	1.01	0.91	1.09	0.95	1.05
			50	0.72	1.30	0.77	1.23	0.77	1.23
10 301	6°	3.2	5	1.13	0.87	1.09	0.91	1.10	0.90
			30	0.99	1.02	0.95	1.05	0.97	1.03
			50	0.72	1.30	0.73	1.27	0.74	1.26
10 221	10°	3.8	5	1.13	0.87	1.09	0.91	1.09	0.91
			25	0.99	1.02	0.95	1.05	0.95	1.05
			40	0.75	1.27	0.76	1.24	0.75	1.25
10 254	10°	3.8	5	1.13	0.87	1.10	0.90	1.08	0.92
			25	0.99	1.04	0.98	1.02	0.96	1.04
			40	0.76	1.25	0.79	1.21	0.79	1.21
10 302	10°	3.8	5	1.15	0.85	1.11	0.89	1.11	0.89
			25	0.99	1.02	0.93	1.07	0.92	1.08
			40	0.72	1.30	0.70	1.30	0.70	1.30
10 308	10°	3.8	5	1.15	0.85	1.11	0.89	1.11	0.89
			25	0.99	1.02	0.96	1.04	0.95	1.05
			40	0.72	1.30	0.74	1.26	0.74	1.26
10 301	10°	3.8	5	1.15	0.85	1.07	0.93	1.09	0.91
			25	0.99	1.02	0.93	1.07	0.93	1.07
			40	0.72	1.30	0.72	1.28	0.72	1.27
8 675	10°	3.8	5	1.16	0.84	1.14	0.86	1.12	0.88
			25	0.99	1.02	0.97	1.03	0.96	1.04
			35	0.82	1.20	0.82	1.18	0.80	1.20

In Tables 9, 10, and 11 are given the calculated vertical wheel loads on straight track and on the outer and inner rails of the curves for all the locomotives with the exception of Locomotive 10301 and for the cars at several speeds, as found by the method just described. It will be noted that the distribution of load on straight track frequently differs from the nominal loads.

It is also seen that in many cases there are transfers of load from wheel to wheel, sometimes quite marked though not as strongly as in the tests of the locomotive reported in the third progress report.

The vertical bending stresses will now be considered for each locomotive and some discussion made of the results.

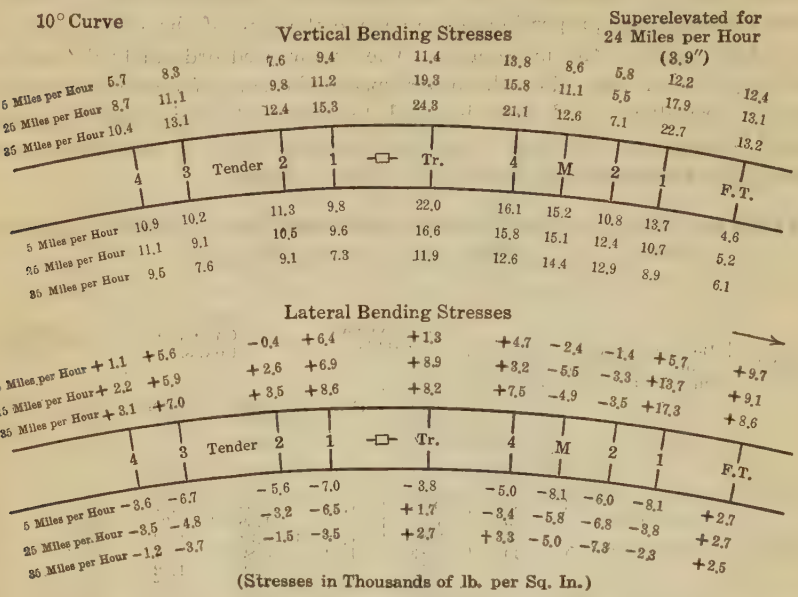


FIG. 31.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 8675 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The results found with Locomotive 8675 will enable some comparison to be made of this Mikado type locomotive with others of that type and the other heavy steam locomotives used in the tests reported in the third progress report, and will better enable comparisons of the electric locomotives to be made. Tests with this locomotive were made on straight track and on the 10° curve. The speed on the curve was limited to 35 miles per hour. It will be recalled that all the tests with the Mikado, Mountain, and Santa Fé types of locomotive given in the third progress report showed an excessive increase of load on one inner driver at a speed of 5 miles per hour and even at speeds up to two-thirds of that corresponding to the super-elevation. It is interesting to find that Locomotive 8675 does not show such a marked increase of load transmitted by a single driver. The vertical bending stress in the inner rail of the 10° curve under the main driver at 5 miles per hour (see Fig. 31) was 15 200 lb. per sq. in., as compared with 12 400 lb. per sq. in. on straight track; the rear inner driver developed 16 100 lb. per sq. in. as compared with 14 100 lb. per sq. in. on straight track. As calculated from the stresses developed at 5 miles per hour, the weight on the main driver on straight track was 31 800 lb. (considerably higher than those on the first two drivers), and that on the inner rail of the 10° curve was 39 400 lb. It will be recalled

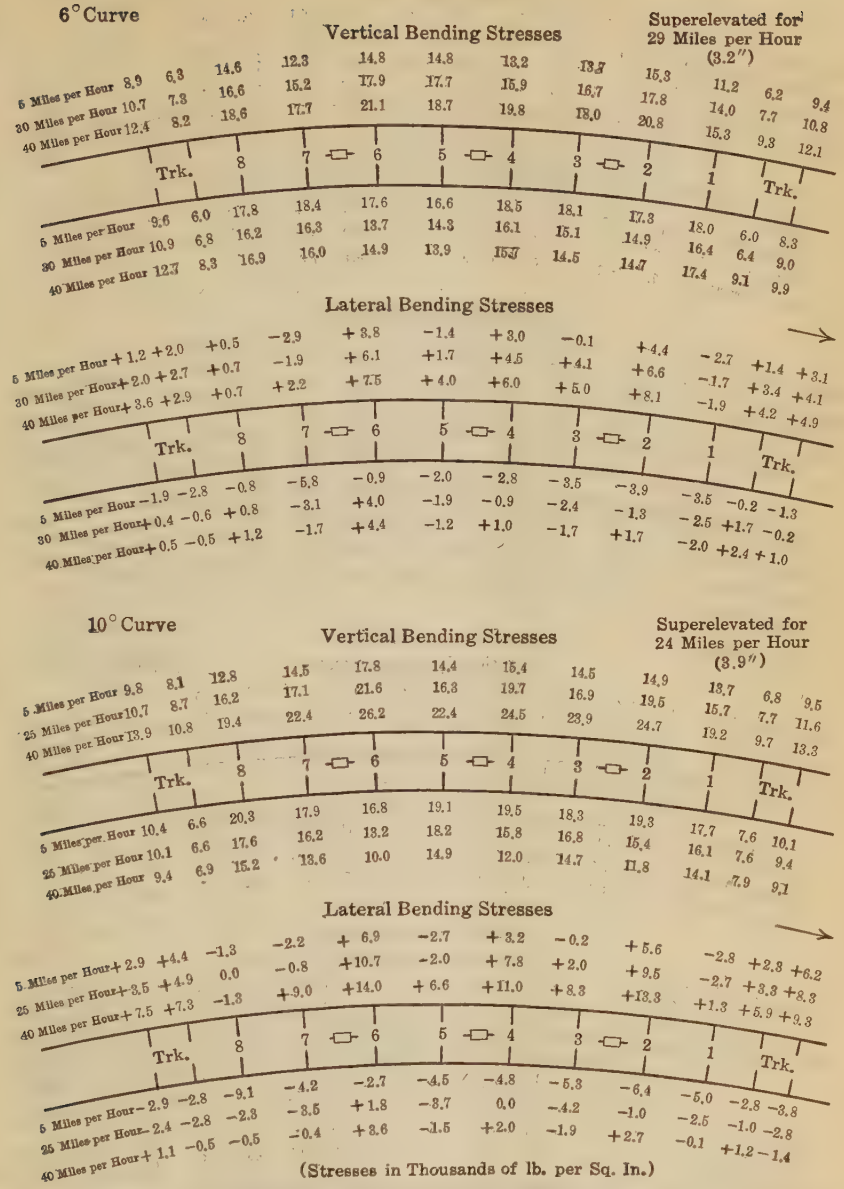


FIG. 32.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 10221 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

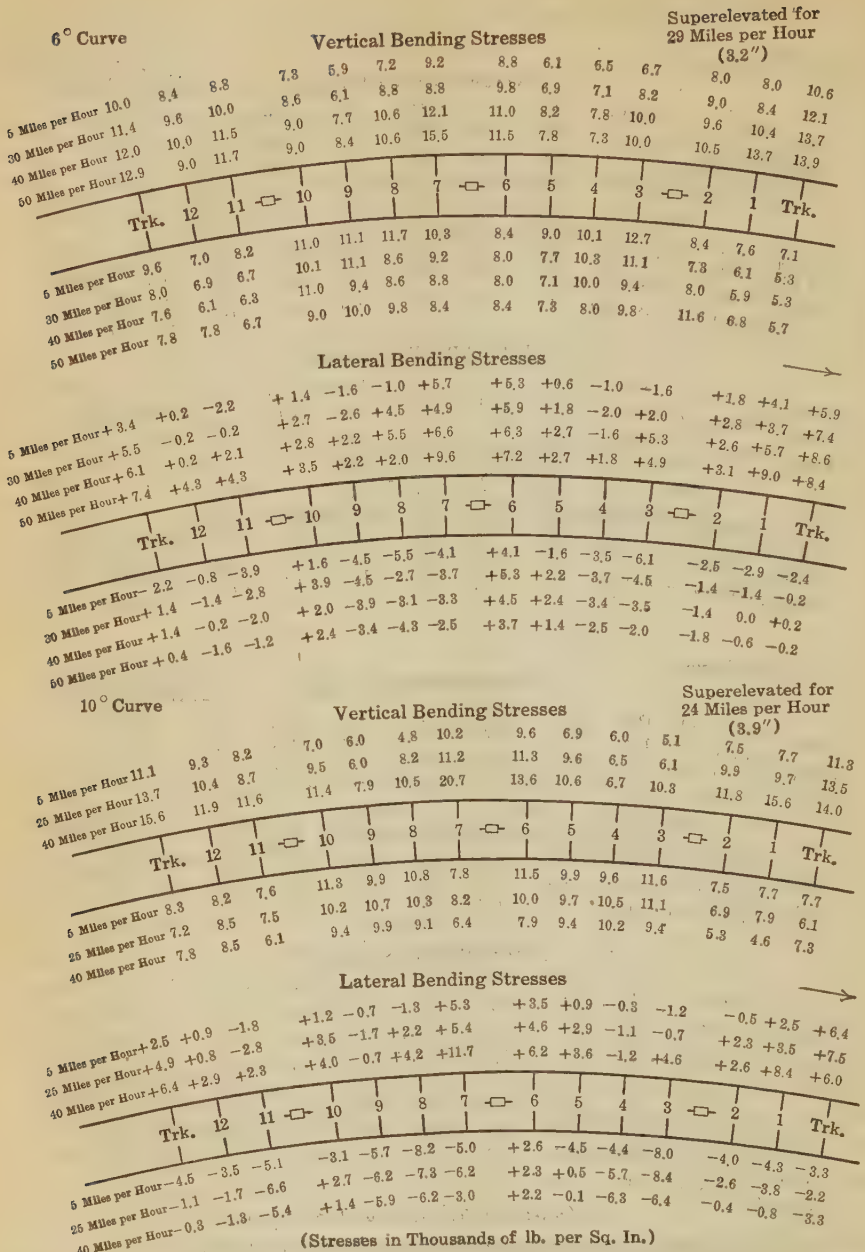


FIG. 33.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 10254 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

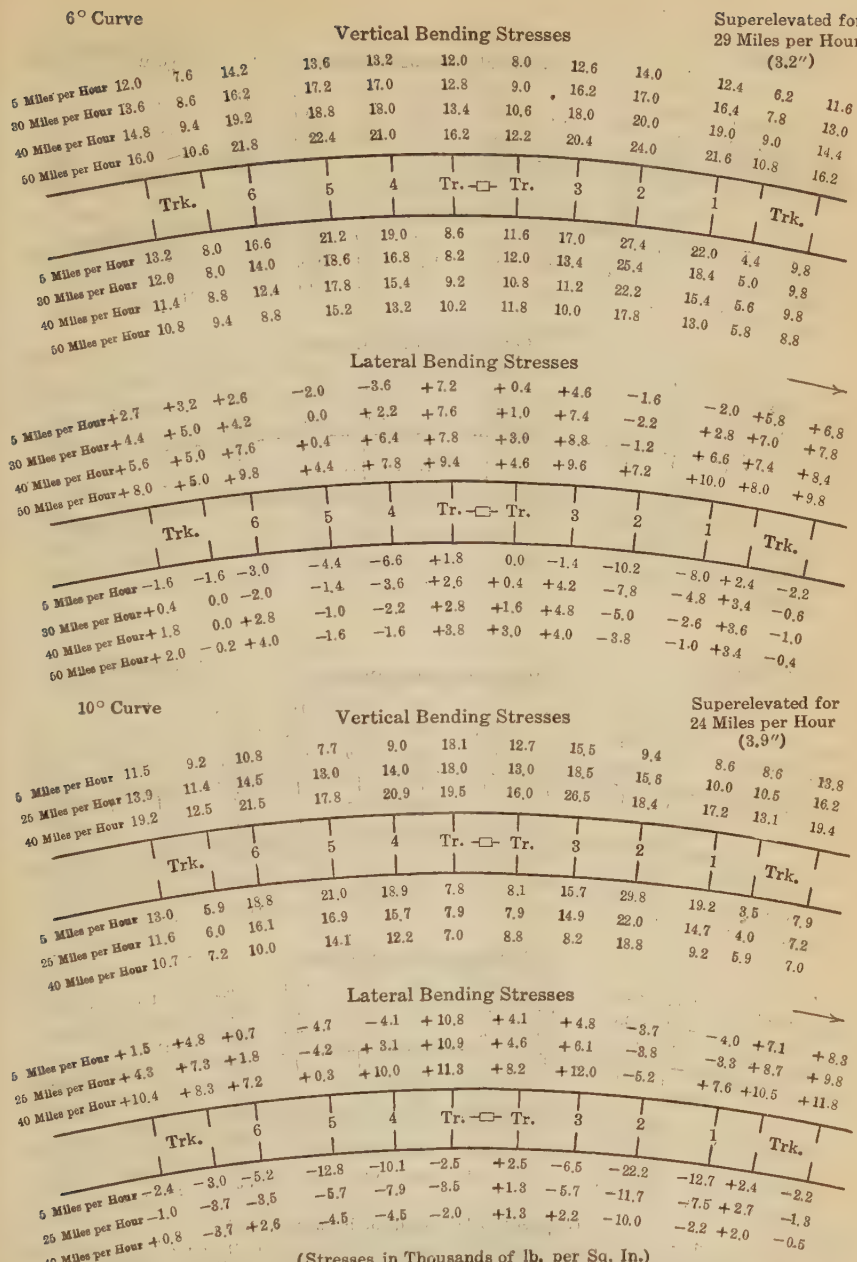


FIG. 34.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 10302 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

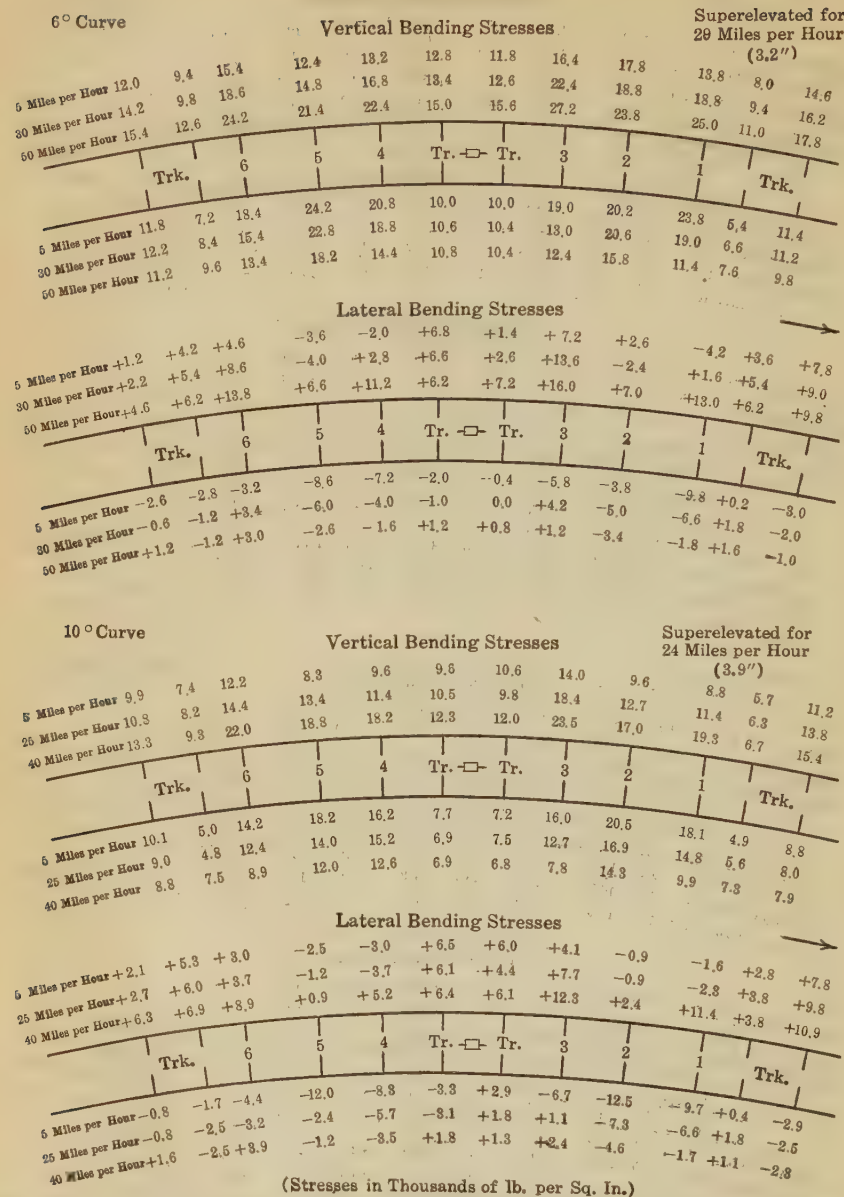


FIG. 35.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 10308 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

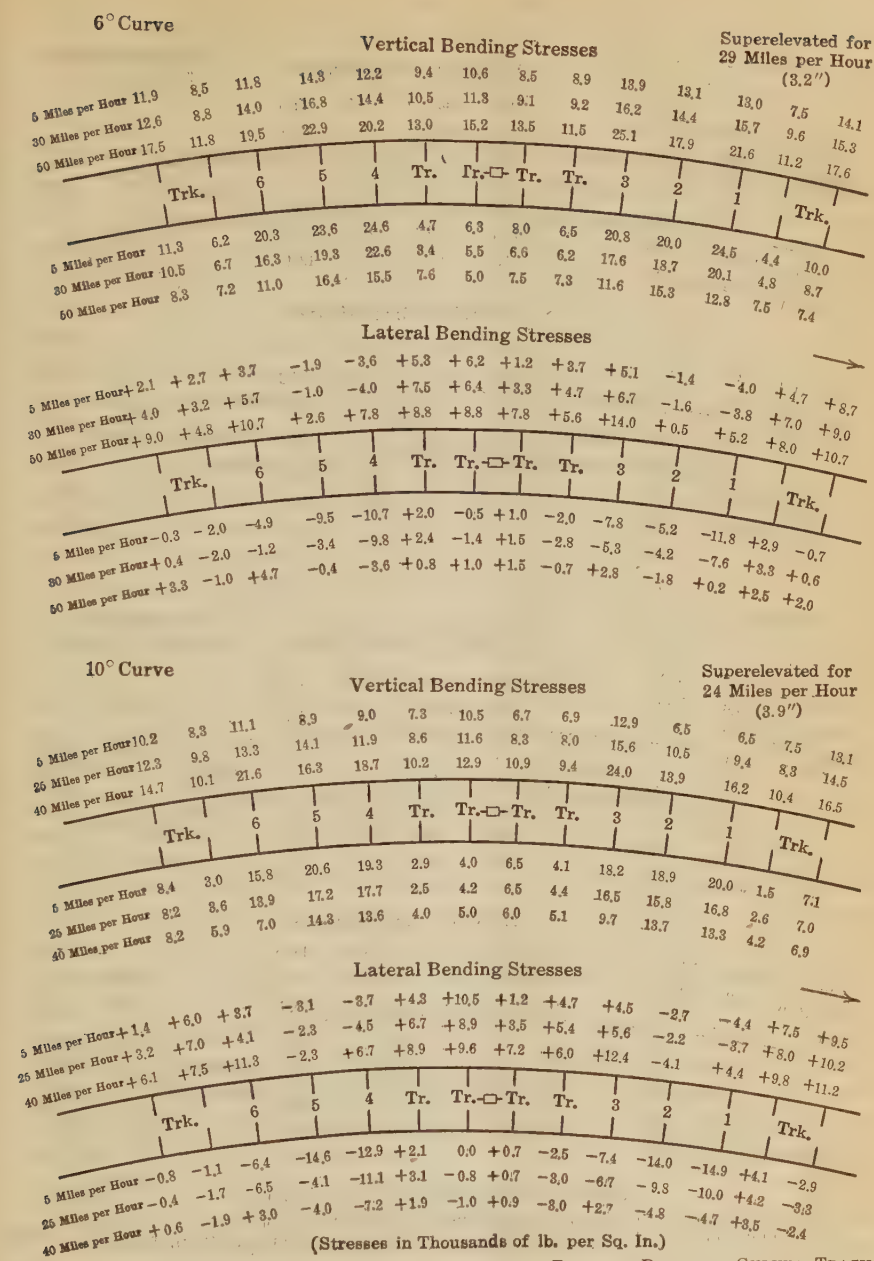


FIG. 36.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOCOMOTIVE 10301 OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

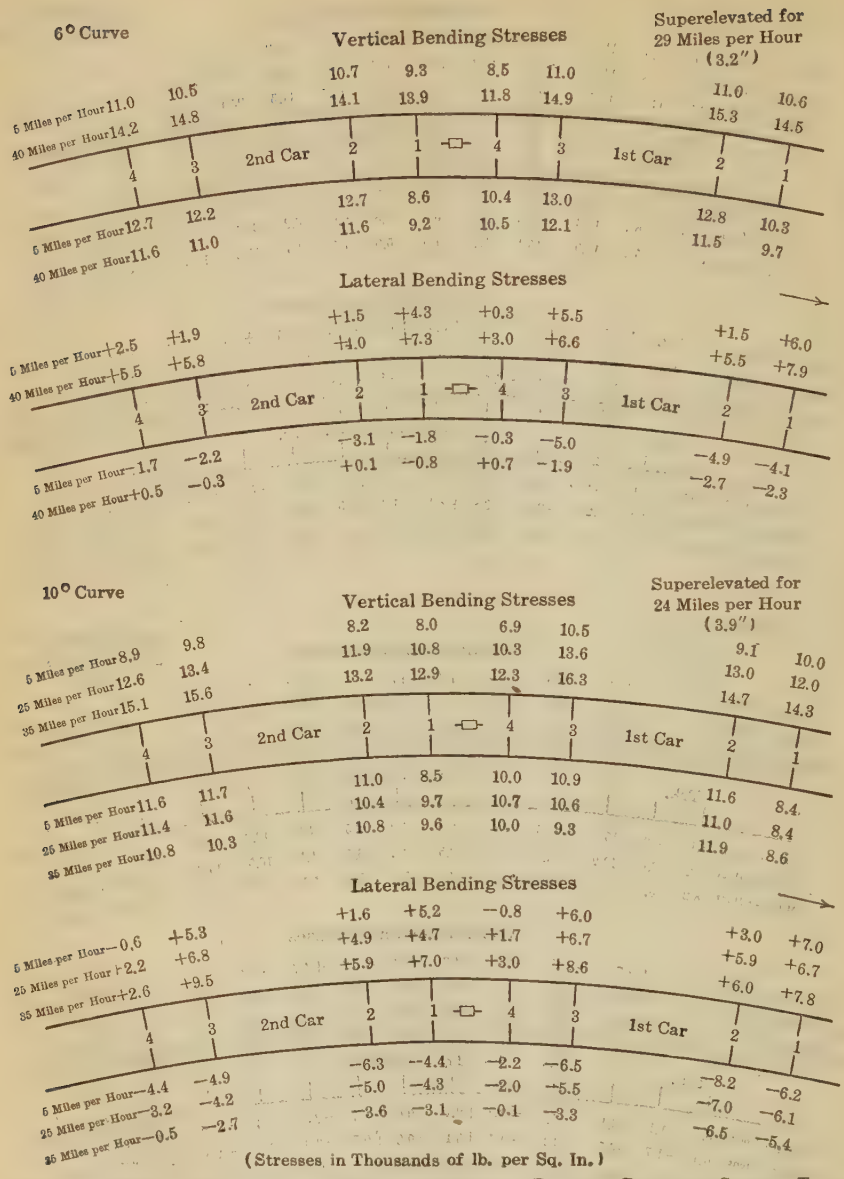

6° Curve
Vertical Bending Stresses
Superelevated for 29 Miles per Hour (3.2″)
5 Miles per Hour 11.0 10.5 10.7 9.3 8.5 11.0 11.0 10.6
40 Miles per Hour 14.2 14.8 14.1 13.9 11.8 14.9 15.3 14.5
4 3 2nd Car 2 1 4 3 1st Car 2 1
5 Miles per Hour 12.7 12.2 12.7 8.6 10.4 13.0 12.8 10.3
40 Miles per Hour 11.6 11.0 11.6 9.2 10.5 12.1 11.5 9.7
Lateral Bending Stresses
5 Miles per Hour +2.5 +1.9 +1.5 +4.3 +0.3 +5.5 +1.5 +6.0
40 Miles per Hour +5.5 +5.8 +4.0 +7.3 +3.0 +6.6 +5.5 +7.9
4 3 2nd Car 2 1 4 3 1st Car 2 1
5 Miles per Hour −1.7 −2.2 −3.1 −1.8 −0.3 −5.0 −4.9 −4.1
40 Miles per Hour +0.5 −0.3 +0.1 −0.8 +0.7 −1.9 −2.7 −2.3
10° Curve
Vertical Bending Stresses
Superelevated for 24 Miles per Hour (3.9″)
5 Miles per Hour 8.9 9.8 8.2 8.0 6.9 10.5 9.1 10.0
25 Miles per Hour 12.6 13.4 11.9 10.8 10.3 13.6 13.0 12.0
35 Miles per Hour 15.1 15.6 13.2 12.9 12.3 16.3 14.7 14.3
4 3 2nd Car 2 1 4 3 1st Car 2 1
5 Miles per Hour 11.6 11.7 11.0 8.5 10.0 10.9 11.6 8.4
25 Miles per Hour 11.4 11.6 10.4 9.7 10.7 10.6 11.0 8.4
35 Miles per Hour 10.8 10.3 10.8 9.6 10.0 9.3 11.9 8.6
Lateral Bending Stresses
5 Miles per Hour −0.6 +5.3 +1.6 +5.2 −0.8 +6.0 +3.0 +7.0
25 Miles per Hour +2.2 +6.8 +4.9 +4.7 +1.7 +6.7 +5.9 +6.7
35 Miles per Hour +2.6 +9.5 +5.9 +7.0 +3.0 +8.6 +6.0 +7.8
4 3 2nd Car 2 1 4 3 1st Car 2 1
5 Miles per Hour −4.4 −4.9 −6.3 −4.4 −2.2 −6.5 −8.2 −6.2
25 Miles per Hour −3.2 −4.2 −5.0 −4.3 −2.0 −5.5 −7.0 −6.1
35 Miles per Hour −0.5 −2.7 −3.6 −3.1 −0.1 −3.3 −6.5 −5.4
(Stresses in Thousands of lb. per Sq. In.)

FIG. 37.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK. LOADED FREIGHT CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

that on the 7½° curve of the Delaware, Lackawanna and Western Railroad, with 105-lb. rail, a vertical bending stress of 18 900 lb. per sq. in. was developed under the third driver of the Mikado type locomotive as compared with 11 100 lb. per sq. in. on straight track. On the 90-lb. rail of the Atchison, Topeka and Santa Fé Railway, a vertical bending stress of 26 700 lb. per sq. in. occurred under the third inner driver of the Mountain type locomotive at 5 miles per hour on the 10° curve and one of 19 700 lb. per sq. in. on the 6° curve, as compared with 13 300 lb. per sq. in. on straight track. Similarly, with the Santa Fé type locomotive a vertical bending stress of 29 200 lb. per sq. in. occurred at the fourth inner driver, as compared with 12 200 lb. per sq. in. on straight track. Why a large increase of load should not be found under one of the drivers of Locomotive 8675 on the inner rail at the lower speeds is not known. It will be shown in Article 13 that the lateral bending stresses in the inner rail for a speed of 5 miles per hour are smaller under all four drivers of this locomotive than were found with the Mikado, Mountain, and Santa Fé types of locomotive used in the earlier tests on curved track. Whether a difference lies in the design of the springs and equalizing system and in other features of the construction that would give a better division of load is not known, but it is evident that in some way the adjustability of this locomotive on the 10° curve at the lower speeds was superior to that of other steam locomotives that have been tested. For the outer rail of the 10° curve, it should be noted that the greatest vertical bending stress under a driver occurs under the first outer driver at 35 miles per hour (the lateral bending stresses showing that this driver at this speed contributes more largely to changing the direction of the locomotive than the track wheel), that the second pair of drivers carries less load than the others, and that at all speeds one or the other of the trailers gives a higher vertical bending stress than is to be found under any driver, the greatest vertical bending stress being 24 300 lb. per sq. in. at a speed of 35 miles per hour.

A characteristic of the vertical bending stresses in Locomotive 10221 (see Fig. 32) is the uniformity of vertical bending stress in either the outer or the inner rail at any speed under all the drivers of the locomotive, the values at a given speed generally not varying more than 2 000 lb. per sq. in. from the average for all the drivers. This is true for both 6° curve and 10° curve. There is then little shifting of load from axle to axle or even from wheel to wheel other than that due to the combined effect of inclination of track and centrifugal force. The spacing of the drivers (10 ft. 6 in. and 11 ft.) is such as not to give much reduction in stress under a driver by reason of the proximity of adjoining drivers, the vertical bending stress thus being relatively high considering the loads carried by the drivers.

The vertical bending stresses developed with Locomotive 10254 (see Fig. 33) were generally much smaller than those found with the other locomotives. The stresses on straight track were especially low. A very even distribution of load among the drivers, as indicated by the vertical bending stresses in rail, is generally characteristic. The exception to the uniformity is found in

TABLE 9.—Calculated Wheel Loads That Would Produce Statically the Measured Vertical Bending Stresses for the Locomotives of the Chicago, Milwaukee and St. Paul Railway.

Speed, in miles per hour.	Rear End Truck.		Driver.								Head End Truck.	
	2.	1.	8.	7.	6.	5.	4.	3.	2.	1.	2.	1.
					Locomotive 10221.							
Straight Track:												
Nominal loads	15 600	15 400	28 300	27 200	29 800	27 600	25 100	27 300	28 700	29 600	15 000	15 100
5	16 300	17 800	28 500	26 600	26 800	27 000	26 300	26 500	26 700	27 400	18 400	16 400
25	17 100	18 600	28 900	27 700	27 100	27 400	26 600	27 000	26 600	28 800	19 600	17 300
40	17 700	19 500	30 100	28 400	28 800	28 700	27 400	28 000	27 900	28 600	19 600	17 900
6° Curve:												
5 Inner	17 200	18 200	31 900	31 200	29 900	28 500	31 200	30 700	29 800	31 900	17 900	15 300
Outer	16 000	17 300	26 200	21 700	24 900	24 800	22 700	23 300	25 300	21 600	16 400	16 500
30 Inner	19 200	19 300	29 500	27 600	23 700	24 400	26 900	25 800	25 800	29 400	18 100	16 300
Outer	19 100	20 100	30 100	26 500	30 100	29 900	27 300	28 400	29 800	26 700	20 000	19 200
40 Inner	22 400	22 400	31 000	27 400	25 300	23 900	26 300	24 900	25 600	31 600	22 900	18 500
Outer	22 000	22 700	33 900	30 800	35 000	32 200	33 300	31 200	34 400	29 700	23 400	21 700
10° Curve:												
5 Inner	17 300	18 700	33 200	28 600	27 000	29 800	30 500	29 100	30 300	30 000	19 200	17 000
Outer	16 500	18 400	22 700	23 100	27 200	23 200	24 100	23 000	23 500	23 600	16 800	15 700
25 Inner	16 700	17 800	28 600	25 700	21 800	27 800	25 200	26 300	24 700	27 200	18 600	16 000
Outer	18 100	20 700	28 000	27 400	32 900	26 700	30 300	27 200	30 200	27 400	19 400	19 100
40 Inner	15 700	17 300	25 500	20 500	18 100	22 500	19 500	22 500	19 500	24 000	18 300	15 500
Outer	23 100	25 600	34 000	35 600	40 300	36 000	38 400	37 800	38 400	33 300	23 900	22 100

TABLE 9.—(*Continued.*)

Speed, in miles per hour.	Rear End Truck.	Driver.											Head End Truck.
	1.	12.	11.	10.	9.	8.	7.	6.	5.	4.	3.	2.	1.	1.
						Locomotive 10254.								
Straight Track:														
Nominal loads...	15 000	18 500	18 600	23 700	23 700	22 200	22 900	22 200	23 200	23 700	23 300	17 900	19 000	16 300
5.....	16 600	17 700	20 900	22 800	21 700	21 900	23 700	22 600	21 700	21 200	22 800	21 300	18 500	17 600
25.....	16 100	17 100	20 000	21 900	20 700	21 300	22 800	21 800	20 700	20 600	22 200	20 500	17 800	17 700
40.....	17 200	18 300	21 300	22 800	21 800	22 200	23 300	22 800	21 800	21 300	23 000	22 500	18 400	17 700
60.....	19 100	21 300	24 300	26 600	25 700	25 800	26 900	27 700	26 600	25 600	25 800	24 400	20 700	19 000
6° Curve:														
5 Inner.....	17 500	16 900	21 500	26 600	27 100	27 900	26 700	24 200	24 200	25 600	27 800	21 600	14 700	14 500
Outer.....	18 100	19 100	19 600	19 200	17 400	18 900	21 600	21 200	17 300	17 400	18 000	19 300	17 700	19 300
30 Inner.....	14 800	15 600	18 500	23 800	25 300	22 900	23 800	22 300	21 500	24 000	24 700	18 600	14 000	9 800
Outer.....	20 300	20 300	21 500	21 600	18 600	21 500	21 500	23 100	19 500	19 500	21 000	21 700	19 100	21 000
50 Inner.....	14 900	16 600	18 400	22 400	23 700	23 700	22 600	22 000	20 300	21 200	22 900	24 000	16 000	12 900
Outer.....	22 400	21 000	26 600	24 800	24 600	27 300	33 400	28 600	23 000	22 000	24 500	26 500	27 400	25 300
10° Curve:														
5 Inner.....	16 300	18 200	21 000	26 600	25 400	26 400	24 200	27 900	25 900	25 400	26 800	21 500	17 400	15 300
Outer.....	19 900	19 700	20 400	18 700	17 300	16 200	22 900	22 500	18 400	17 000	16 100	18 500	17 300	19 500
25 Inner.....	14 800	18 100	20 300	25 200	25 900	25 700	24 300	26 000	25 300	25 900	25 900	19 400	16 900	13 100
Outer.....	24 000	22 600	23 100	23 600	19 800	22 300	26 600	26 900	23 600	19 900	20 600	23 500	21 600	23 600
40 Inner.....	15 200	17 700	18 000	22 900	23 400	22 600	20 200	21 900	23 200	23 800	22 400	15 800	12 000	13 200
Outer.....	27 800	26 700	29 200	29 500	26 800	30 200	42 200	34 600	28 600	23 700	27 800	29 600	30 900	26 800

TABLE 10.—Calculated Wheel Loads That Would Produce Statically the Measured Vertical Bending Stresses for Locomotives of the Chicago, Milwaukee and St. Paul Railway.

Speed, in miles per hour.	Rear End Truck.		Driver.			Trailer.		Driver.			Head End Truck.	
	2.	1.	6.	5.	4.	2.	1.	3.	2.	1.	2.	1.
Straight track:						Locomotive 10302.						
Nominal loads	17 000	17 000	33 000	29 900	31 400	25 300	25 000	31 800	35 300	34 000	14 500	14 500
5	20 000	18 800	28 600	31 500	32 000	25 000	21 400	30 600	37 500	28 600	17 500	17 500
25	20 300	19 700	29 300	31 800	32 200	25 300	22 100	31 000	38 600	29 300	17 400	18 600
40	21 700	20 800	30 800	33 100	33 200	26 400	23 400	32 200	39 600	31 300	19 800	20 400
60	26 600	26 800	35 800	37 500	37 800	31 300	28 500	36 400	42 600	33 900	23 300	22 200
6° Curve:												
5 Inner	22 300	21 000	31 500	37 700	35 100	22 100	25 400	34 900	46 600	38 600	16 800	17 300
Outer	20 100	19 000	26 300	25 700	25 800	23 600	19 100	24 200	25 500	23 400	16 400	19 000
30 Inner	20 300	19 700	27 300	33 200	31 300	20 800	24 600	29 300	42 300	33 400	16 500	17 100
Outer	22 900	21 600	30 400	32 000	32 400	26 400	22 000	30 400	31 300	30 300	20 100	21 800
50 Inner	18 500	19 600	19 700	26 800	25 800	22 100	23 300	22 600	30 300	24 600	14 600	15 300
Outer	27 400	27 000	40 100	41 500	40 600	33 600	28 800	39 800	43 400	40 100	27 300	27 700
10° Curve:												
5 Inner	22 400	19 300	35 100	39 200	36 000	20 900	20 900	33 800	50 500	36 100	14 800	14 800
Outer	20 100	20 500	21 600	17 000	21 200	33 000	28 300	29 500	20 200	19 100	19 800	22 900
25 Inner	20 200	18 000	30 200	32 000	30 200	19 800	19 800	30 400	38 400	28 000	13 500	13 300
Outer	24 700	25 900	29 000	26 600	29 900	34 900	30 200	35 600	30 300	23 400	23 700	27 100
40 Inner	18 400	17 400	21 100	26 000	24 100	17 100	18 500	20 000	31 200	20 000	14 400	12 900
Outer	33 300	31 400	41 200	36 800	41 900	40 500	37 700	49 000	38 100	35 600	31 100	33 300

TABLE 10.—(*Continued*).

Speed, in miles per hour.	Rear End Truck.		Driver.			Trailer.		Driver.			Head End Truck.	
	2.	1.	6.	5.	4.	2.	1.	3.	2.	1.	2.	1.
					Locomotive 10308.							
Straight Track:												
Nominal loads	16 600	16 600	33 400	33 200	32 700	23 300	22 700	30 500	37 500	37 300	15 100	15 100
5	18 300	17 000	29 400	31 800	32 500	24 600	22 800	33 000	34 600	30 800	19 300	19 900
40	20 000	20 200	31 700	34 300	33 900	26 000	23 600	33 500	35 000	31 000	20 200	21 300
60	20 800	22 500	33 800	37 100	36 000	27 300	25 700	36 600	37 400	33 000	23 100	22 700
6° Curve:												
5 Inner	19 300	19 000	32 400	40 300	36 700	23 100	22 600	33 600	35 700	38 000	17 800	18 800
Outer	19 100	20 600	26 700	23 100	24 700	24 600	24 300	29 800	30 400	25 600	19 300	22 500
30 Inner	19 800	19 800	28 100	37 500	33 800	23 000	21 600	25 800	34 000	32 000	18 100	18 400
Outer	25 100	32 900	35 400	37 600	32 200	27 200	26 500	38 300	34 200	33 100	22 900	25 400
50 Inner	18 600	20 300	25 200	30 300	26 900	21 900	20 900	23 400	26 200	21 400	16 600	15 900
Outer	25 900	28 600	40 500	38 800	40 000	31 700	33 400	46 800	43 100	42 800	27 500	28 600
10° Curve:												
5 Inner	20 600	18 300	32 100	39 400	36 600	22 900	22 300	37 000	43 500	38 900	19 200	19 000
Outer	20 600	21 000	26 800	20 400	23 200	24 000	26 700	31 000	23 400	21 400	17 400	21 700
25 Inner	18 400	17 500	27 800	31 600	33 300	20 900	22 100	30 100	36 100	32 400	18 700	17 400
Outer	22 800	23 700	32 200	30 400	28 200	26 600	27 400	39 200	30 600	27 200	20 500	26 600
40 Inner	18 400	19 800	22 300	26 800	28 000	19 300	18 000	20 800	29 300	24 000	19 600	17 200
Outer	28 500	29 500	47 200	43 400	42 400	33 300	34 200	50 400	41 300	42 000	24 900	30 500

the outer rail under Driver No. 7 (the leading driver of the second half of the locomotive) for the highest speeds on both the 6° and 10° curves, where the stress is considerably greater than under the other drivers on either rail, 15 500 lb. per sq. in. on the 6° curve, and 20 700 lb. per sq. in. on the 10° curve. It will be shown that the lateral bending stress in the outer rail under this driver (11 700 lb. per sq. in. on the 10° curve at 40 miles per hour) is much greater than that under the outer drivers, and that the flange of this driver is the principal factor in the turning action of the second unit of drivers in traversing the curve. It seems possible that the springs and equalizing system are in some way affected in their action by this strong lateral pressure.

TABLE 11.—Calculated Wheel Loads That Would Produce Statically the Measured Vertical Bending Stresses for Locomotive 8675 and Loaded Freight Cars of the Chicago, Milwaukee and St. Paul Railway.

Speed, in miles per hour.	Trailer.	Driver.				Head End Truck.
		4.	Main.	2.	1.	1.
Straight Track:						
Nominal loads	28 500	30 500	30 500	29 300	29 200	12 000
5	25 200	31 300	31 800	27 600	28 200	16 000
25	27 600	33 800	33 800	29 200	29 600	17 300
40	27 200	35 200	35 300	30 800	31 200	18 400
10° Curve:						
5 Inner	35 600	36 200	39 400	31 200	29 600	10 500
5 Outer	19 400	27 900	23 600	20 000	25 800	21 400
25 Inner	27 600	35 000	36 800	32 400	25 600	10 900
25 Outer	31 400	33 300	29 600	23 100	36 500	23 400
35 Inner	20 200	29 100	34 400	31 900	23 000	11 800
35 Outer	39 600	43 400	35 200	27 600	43 900	24 800

	Second Car.				First Car.			
	4.	3.	2.	1.	4.	3.	2.	1.
Straight Track:								
Nominal loads	19 000	19 000	18 800	18 800	19 700	19 700	19 000	19 000
5	18 800	18 500	19 100	19 600	19 600	19 500	19 000	18 700
40	20 100	20 100	21 700	23 700	23 700	22 000	22 100	22 000
° Curve:								
5 Inner	19 900	19 400	20 200	18 800	20 900	21 000	20 500	20 000
5 Outer	17 300	16 700	17 600	18 600	17 800	17 900	18 400	20 700
40 Inner	18 100	17 600	18 900	19 400	22 800	19 800	18 800	19 600
40 Outer	22 800	23 200	23 800	27 000	24 800	24 300	25 400	28 000
10° Curve:								
5 Inner	15 800	19 500	22 400	22 000	20 000	19 900	22 000	25 200
5 Outer	16 000	17 000	15 400	17 600	16 700	18 600	17 200	20 400
25 Inner	20 200	20 300	19 500	21 800	23 200	20 000	20 000	20 000
25 Outer	22 500	23 400	22 000	24 200	24 000	24 500	23 900	25 600
35 Inner	18 900	18 300	19 900	21 500	21 700	17 900	21 200	20 100
35 Outer	26 700	27 400	24 900	28 600	28 500	29 300	27 400	30 300

Locomotive 10302 shows greater diversity in the vertical bending stress under the several drivers, both on straight track and on the 6° and 10° curves, than the two electric locomotives just considered. On straight track

Driver No. 2 on both rails carries a considerably greater load than the adjacent drivers, 30% more as calculated from the stresses. The vertical bending stress in the inner rail under this driver for a speed of 5 miles per hour is 27 400 lb. per sq. in. on the 6° curve and 29 800 lb. per sq. in. on the 10° curve (the latter being indicative of a wheel load of 49 400 lb.), both vertical bending stresses being relatively very great, and that in the outer rail on the 6° curve under the corresponding driver for a speed of 50 miles per hour is 24 000 lb. per sq. in., while on the 10° curve the highest vertical bending stress in the outer rail, at 40 miles per hour (26 500 lb. per sq. in.), is shifted to Driver No. 3. The middle drivers of the second half of the locomotive do not develop as great stresses as the corresponding drivers of the first half.

For Locomotive 10308 the loads are more nearly equally divided among the drivers, as shown by the vertical bending stresses on straight track, than is the case with Locomotive 10302. At 5 miles per hour the high vertical bending stresses are in the inner rail under the middle driver of each unit of drivers (Nos. 2 and 5) for the 10° curve, the greatest being 20 500 lb. per sq. in., and under Nos. 1 and 5 for the 6° curve, 24 000 lb. per sq. in. At 50 miles per hour on the 6° curve the highest stresses are in the outer rail, 27 200, 25 000, and 24 200 lb. per sq. in., and at 40 miles per hour on the 10° curve, 22 000 and 23 500 lb. per sq. in., the highest stress being under the rear driver of a group. It is seen that the increase of wheel load on curved track, as indicated by the vertical bending stresses, except for Driver No. 2, does not differ greatly from that given by the locomotive in the original form.

For Locomotive 10301 the vertical bending stresses under the drivers on straight track indicate a good division of the load among the drivers. The greatest vertical bending stresses on curved track occur under different wheels; on the inner rail of the 10° curve under Drivers Nos. 1 and 5, at 5 miles per hour, the stresses are 20 000 and 20 600 lb. per sq. in., and on the outer rail under Drivers Nos. 3 and 6, at 40 miles per hour, 24 000 and 21 600 lb. per sq. in.; on the inner rail of the 6° curve under Drivers Nos. 1 and 4, at 5 miles per hour, 24 500 and 24 600 lb. per sq. in., and on the outer rail under Drivers Nos. 3 and 5, at 50 miles per hour, 25 100 and 22 900 lb. per sq. in. Except for the last named, the high vertical bending stresses are accompanied by the greatest of the lateral bending stresses in the rail found at the given speed. It is seen that the increase of wheel load in curved track, as indicated by the vertical bending stresses, is somewhat less than for the locomotive in the original form.

For the loaded freight cars on straight track it was found, as would be expected, that the vertical bending stresses under wheels that are close to wheels of the tender or to those of another car, are smaller than those under wheels that do not have another wheel within a few feet of it on both sides. The same influence of adjoining wheels is found on the two curves regardless of speed (Fig. 37). The vertical bending stresses on the curves are moderate, the greatest being 16 300 lb. per sq. in. on the 10° curve, at 35 miles per hour. It is evident from what has been said that with the four-

wheel truck there is little shifting of load from axle to axle on the curve, or even from wheel to wheel, except what is due to centrifugal force and inclination of track.

13.—Lateral Bending Stresses in Rail on Curved Track and Position of Flanges.—On straight track irregularities in the direction of the movement of wheels and sidewise oscillations produce lateral bending of the rail, as has already been discussed. When a locomotive or a car traverses curved track, as may be expected from analytical considerations, additional forces are applied to the rail in a lateral direction, which set up still greater lateral bending of the rail. The flange of a truck wheel or a leading driver presses against the outer rail, developing a force which changes the direction of movement of the group of wheels connected to the same frame, in the manner already described under Article 10, "The Action of Curved Track". The lateral pressures on the rail given by other wheels of the unit are most generally transmitted to the rail as either pull or push by the friction of the tread on the rail, and may be directed inwardly or outwardly of the curve according to the position in the unit and the relation of the speed to superelevation and centrifugal force and also according to the number of wheels in the unit and even to the degree of stiffness and flexibility of the connection between units.

It may be helpful in studying the lateral bending stresses to show the position of the wheels with respect to the rail that was observed on a curve after the locomotive had come to rest without brakes being applied. In Figs. 38 and 39 is given the position of the wheels of six locomotives relative to the rail; the dimensions are given in inches and represent the distance from the gauge side of the rail to the flange of the wheel. The measurements were made at points a little above the mid-height of the head of the rail. The radial line on the diagrams is drawn at the point where the line taken by the wheel-base appears to be perpendicular to the radius of the curves. The point where this radial line crosses the inner rail may be taken to be the probable center of rotation of the locomotive frame. Here, again, the 6° curve is shown as curving to the right instead of to the left.

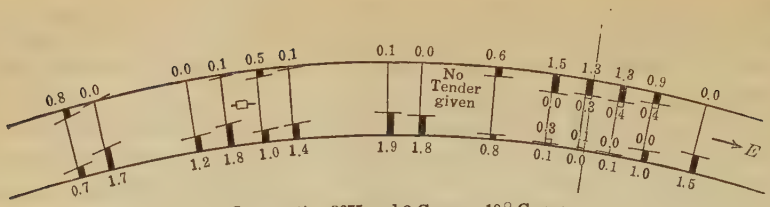
The wheels of Locomotive 8675 (Mikado type) on the 10° curve (see Fig. 38) took positions that are relatively the same as the positions found with the Mikado type locomotive on the 6° and the 7½° curves of the Delaware, Lackawanna and Western Railroad and the Santa Fé type locomotive on the 6° and the 10° curves of the Atchison, Topeka and Santa Fé Railway, reported in the third progress report. The flange of the outer front truck wheel bore against the outer rail, and that of the first outer driver was 0.9 in. away, and yet outward lateral bending of the outer rail was found under this driver at 5 miles per hour. The center of rotation appeared to be under the third inner driver. The flanges of the second, third, and fourth inner drivers were close to the inner rail and the flanges of the corresponding outer drivers were well away from the outer rail and at about the same distance from it. Measurements made on the position of the journals in the boxes showed that the journal of the first outer driver still had all its outward play available so that it was not helping in changing the direction of the locomotive. The

journal of the rear outer driver was at the limiting position in its bearing and, hence, was in position to put outward thrust on the outer rail as the frame was swung outwardly about the center of rotation. Measurements made to find the change in alignment of the rails as the locomotive passed at 2 miles per hour showed movements of the head of the rail in keeping with the positions of the flanges. The impressions on the copper wire laid across the rail and pulled laterally between the passage of wheels showed that the flange of the first outer truck wheel and first driver bore strongly against the outer rail at the speed of 35 miles per hour. This condition is also borne out by the high lateral bending stresses found under the two wheels at this speed. The impression on the copper wire also showed that the first outer wheel of each truck of the tender bore strongly against the outer rail at the speed of 35 miles per hour.

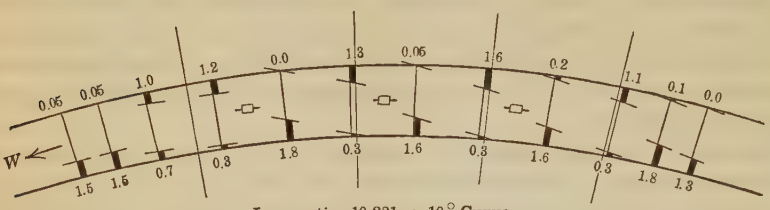
The position of the flanges of Locomotive 10221 on the 10° curve indicates that the outer truck wheels of the first half of the locomotive at the lower speeds exercise the turning action for the frame containing the first four wheels. The first outer drivers of the next three groups of wheels lie close to the outer rail, seemingly close to the position that would permit turning forces to be developed. The impressions of the copper wires, however, show at 25 miles per hour flange contact with the outer rail by the flanges of only the first outer wheel of the front truck and the first outer wheel of the rear truck. No explanation is known for this condition. All the runs on which impressions were taken, nine in number, agree in this particular. It would seem probable that the several groups of wheels have so adjusted themselves on the curve that, with the stiffness of the connection, the flange pressure of the leading truck wheels is the principal agency in changing the direction of the locomotive in traversing the curve.

The flanges of the first three outer wheels of Locomotive 10254, when the locomotive was allowed to come to rest (see Fig. 38), were found to be against the outer rail on both 6° and 10° curves. In the second group the flange of the leading outer driver was about 0.4 in. away from the rail, and the others of this group lay away from the rail. A consideration of the lateral bending stresses described subsequently indicates that at the low speed the flanges of the leading group of wheels act to change the direction of the wheels of the first half of the locomotive. The flange of the first outer driver of the third group of wheels (the leading driver of the second half of the locomotive) is against the outer rail, and the lateral bending stress and the position of the flanges of the fourth group of wheels and the corresponding lateral bending stresses indicate that the flange of the first outer driver of the third group acts at low speeds to change the direction of the second half of the locomotive. At the high speeds the leading outer wheel of the second group participates in the turning action, as indicated by the lateral bending stresses.

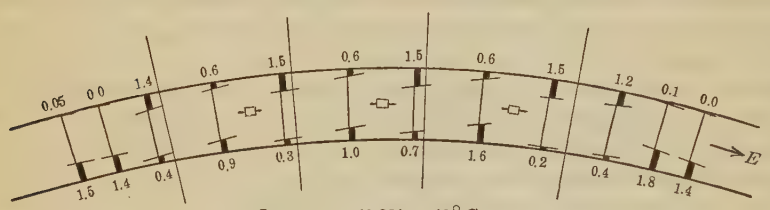
In the three forms of the Westinghouse-Baldwin locomotive, the position of the flanges of the wheels with respect to the rail, when the locomotive was allowed to come to rest, did not differ materially. The flange of the first outer truck wheel was against the outer rail of the 10° curve and that of the second outer truck wheel was close to it. The flanges of all the outer drivers were



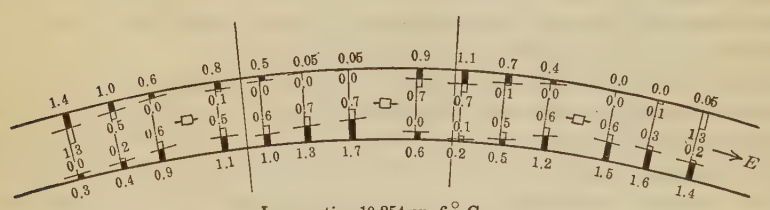
Locomotive 8675 and 2 Cars on 10° Curve



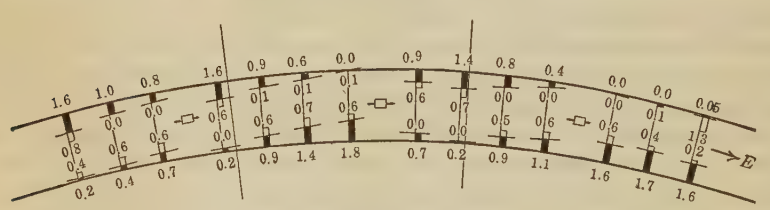
Locomotive 10 221 on 10° Curve



Locomotive 10 221 on 10° Curve



Locomotive 10 254 on 6° Curve

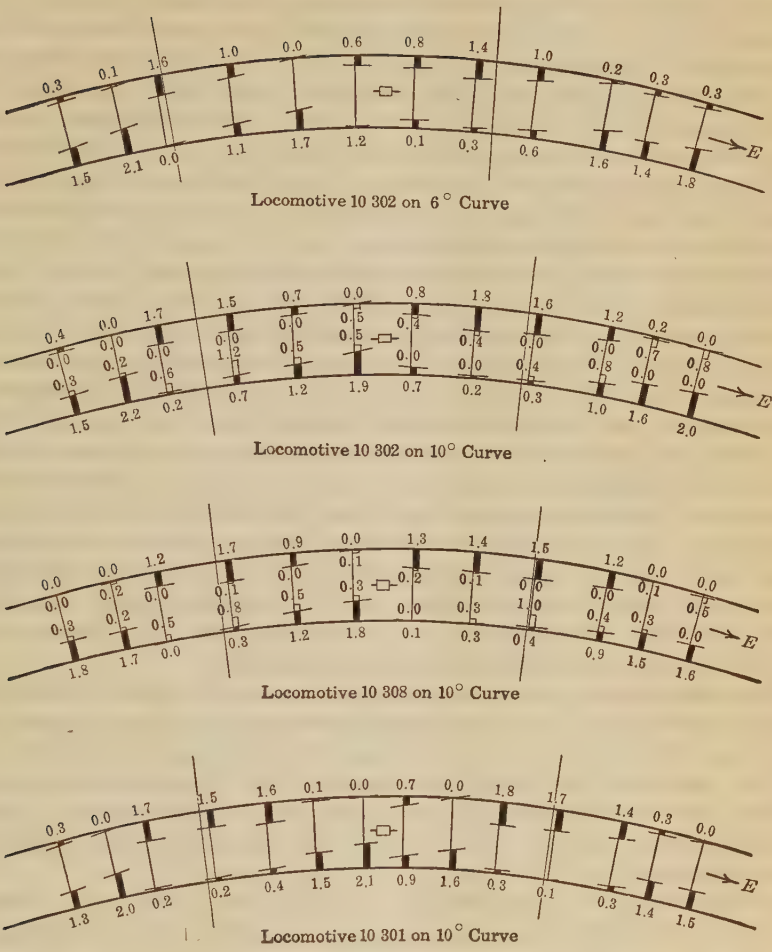


Locomotive 10 254 on 10° Curve

▮ Distance between Flange and Rail in Inches

▯ Journal Play in Inches

FIG. 38.—RELATIVE POSITION OF FLANGES OF WHEELS OF LOCOMOTIVES WITH RESPECT TO THE RAILS OF CURVED TRACK ON THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.



▮ Distance between Flange and Rail in Inches

▯ Journal Play in Inches

FIG. 39.—RELATIVE POSITION OF FLANGES OF WHEELS OF LOCOMOTIVES WITH RESPECT TO THE RAILS OF CURVED TRACK ON THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

away from the outer rail 1 in., or more, the first driver of each group being closer than the last. The wheel following the front group of drivers is also away from the outer rail, except for Locomotive 10301. The wheel in front of the rear group of drivers (and the two wheels of Locomotive 10301) is always against the outer rail, and evidently is the principal agent in changing the direction of the second half of the locomotive. The flange of the first outer wheel of the rear truck is always against the outer rail, and that of the second is close to it. The flanges of the inner drivers are usually close to the inner rail, though not against it. The impressions made with the copper wire on the outer rail at the speed of 25 miles per hour on the 10° curve indicated that the same flanges, and no others, were against the outer rail as when the locomotive came to rest. At 40 miles per hour the flange of the first outer driver of each group bore against the outer rail. On the inner rail the impressions taken at 5 miles per hour indicate that the flanges of the second and third inner drivers of each group were usually close to the rail. On the 6° curve the impressions taken with Locomotive 10308 indicate that at speeds of 30 and 50 miles per hour the flange of the first outer driver of each group of drivers bore against the outer rail; this was the only locomotive of this type for which impressions were taken on the 6° curve. In view of all the observations, it is evident that the wheels of the three forms of the Westinghouse-Baldwin locomotive took much the same position on the curves and that the modifications of the original form cannot have done more than modify the intensity of pressures. Such changes may be indicated by the lateral bending stresses in the rail. The high values of the lateral bending stress in the outer rail found under the rear driver of both halves of the locomotives at the highest speed is an indication that at high speeds the flange of these drivers may bear against the outer rail and thus that flange wear may occur on these drivers, as has been reported by the railroad company.

The impressions with copper wire made under the wheels of the loaded freight cars on the 6° curve at a speed of 40 miles per hour indicated that the flange of the first outer wheel of each truck bore against the outer rail, as also did that of the second outer wheel of the leading truck of the first car and the second outer wheel of the rear truck of the second car. These positions agree in general with the position of the wheels measured after the cars had come to rest.

In considering the lateral bending of the rails of curved track, it should be kept in mind that as the beam strength of the rail in a lateral direction is much less than that which resists bending in a vertical direction (the section modulus about a vertical axis being only about one-fifth of that about a horizontal axis), large lateral bending stresses will be developed by a lateral bending moment relatively much smaller than the vertical bending moment that causes vertical bending stresses of the same magnitude. The value of the lateral bending stress that is reported is the amount by which the stress in one edge of the base of rail exceeds the mean of the stresses in the two edges (the latter being the stress called the vertical bending stress). For both outer and inner rail the stress is called positive if it tends to increase the curvature in the rail and negative if it tends to straighten the rail. The maximum fiber stress

in the base of rail will be equal, of course, to the numerical sum of the vertical bending stress and the lateral bending stress.

It will be recalled that in locomotives of the Mikado type it was reported in the third progress report that the lateral force necessary to change the direction of the locomotive in traversing a curve was developed by the outer rail against the flange of the outer truck wheel and generally also of the first driver, and that the pressure against the flange of the first driver increased considerably with an increase in speed. The outward lateral bending stresses in the rail under these two wheels thus developed by these lateral turning forces are among the greatest lateral bending stresses found. With Locomotive 8675 the lateral bending stress in base of rail under the first outer driver (see Fig. 31) ranged from 5 700 lb. per sq. in. at 5 miles per hour, to 17 300 lb. per sq. in. at 35 miles per hour. The latter stress is higher than was obtained on the outer rail of the 10° curve at a speed of 40 miles per hour with any of the electric locomotives. The lateral bending stress in the outer rail under the truck wheel remains nearly constant at the three speeds, which indicates that the restrictive action of the connection of the leading truck to the frame of the drivers is limited and that at the highest speed the first outer driver participates strongly in the guiding action. At the highest speed the flange of the first outer driver cut off the copper wire at almost every attempt to obtain a record. As is usual with this type of locomotive, the rear driver and the trailer also produced outward bending in the outer rail at all speeds. On the inner rail the higher lateral bending stresses at a speed of 5 miles per hour were under the first and third drivers, 8 100 lb. per sq. in. at each point. These stresses are relatively considerably smaller than those found with Mikado, Mountain, and Santa Fé types of locomotive reported in the third progress report. They are about the same as those developed with the General Electric freight and passenger locomotives and less than those found with Westinghouse-Baldwin locomotives. It has already been stated that this steam locomotive did not develop an excessive increase of load on one of the inner drivers at the lower speeds, as was generally the case with the locomotives having eight and ten drivers reported in the third progress report.

With Locomotive 10221 the lateral bending stresses in the inner rail are relatively small (see Fig. 32), the greatest being 9 100 lb. per sq. in. under one driver on the 10° curve and 5 800 lb. per sq. in. on the 6° curve at 5 miles per hour, all the other values being considerably smaller. In the outer rail the stresses increase with an increase in speed. At 40 miles per hour the greatest lateral bending stress in the outer rail on the 6° curve is 8 100 lb. per sq. in. and on the 10° curve, 14 000 lb. per sq. in. Strangely the higher lateral bending stresses in the outer rail on both curves occur under the rear driver of each group except for the last group. As was stated in the discussion of the position of the flanges of this locomotive with respect to the rail on curved track and of the impressions given on copper wire, it is not clear how forces do act to change the direction of this locomotive at other than the first pair of drivers.

With Locomotive 10254 the lateral bending stresses in the inner rail are small (see Fig. 33), the greatest being 8 400 lb. per sq. in. on the 10° curve

and 6 100 lb. per sq. in. on the 6° curve. In the outer rail the lateral bending stresses are also low, the greatest at 50 miles per hour on the 6° curve being 9 600 lb. per sq. in. and at 40 miles per hour on the 10° curve 11 700 lb. per sq. in., both being under the first driver of the second half of the locomotive, which according to the observed position of the flanges is the principal guiding wheel for the second half of the locomotive. Other than for this driver, the lateral bending stresses in the outer rail are generally small and fairly uniform. The values of the lateral bending stresses, taken in connection with the position of the flanges of the wheels when the locomotive is at rest, already described, indicate that the first three outer wheels are the principal factors in changing direction in the first half of the locomotive and the first outer driver of the second half of the locomotive is the principal factor for the second half. Attention has already been called to the higher vertical bending stress under the last named driver at the higher speeds and to the consequent higher load transmitted to the rail by this driver.

The three forms of the Westinghouse-Baldwin locomotive showed much the same characteristics in the development of lateral bending stresses, and the magnitude of the stresses in general did not differ greatly. (See Figs. 34 to 36.) Locomotive 10302 (the original form) gave one exception to this statement. The lateral bending stress in the inner rail under the second driver at 5 miles per hour was extremely high, 22 200 lb. per sq. in., and this was accompanied by an extremely high vertical bending stress, 29 800 lb. per sq. in. Other than this value, the lateral bending stresses under the wheels of the three locomotives are fairly comparable at all speeds and in both rails of both curves. The higher values of the lateral bending stresses in the inner rail of the 10° curve range from 12 000 to 15 000 lb. per sq. in. and in the outer rail, from 10 000 to 12 000 lb. per sq. in.; in the inner rail of the 6° curve, from 10 000 to 12 000 lb. per sq. in., and in the outer rail, from 11 000 to 16 000 lb. per sq. in. In the outer rail of both curves at the highest speeds, the higher lateral bending stresses in general occur under the rear driver of the two groups, an unexpected result. The higher lateral bending stresses in the outer rail in general do not occur under those drivers that would be expected to participate strongly in changing the direction of the locomotives. In fact, the first outer driver of each half of the locomotive does not participate in the guiding action, which is taken principally by the outer truck wheels ahead of the drivers. The high lateral bending stresses in the outer rail under the rear outer driver of each group of drivers at the higher speeds on both curves may have some relation to the larger flange wear of these wheels reported by the railroad company, although it is difficult to understand why the flanges of these two drivers should be strongly pressed against the rail, and the tests with the copper wire gave no indication of such contact. It may be noted that at the highest speeds the vertical bending stress under these two drivers has also been increased greatly, indicating that the loads carried by these drivers are considerably higher than the average of those carried by the other drivers on the outer rail.

The lateral bending stresses under the wheels of the loaded freight cars on both curves (see Fig. 37) are significant. The stress in the outer rail of

both curves indicates that the flange of the front outer wheel of each truck acts to change the direction of the truck so as to traverse the curve. The lateral bending stresses in the outer rail on both curves at all speeds under all wheels almost without exception act to increase the curvature of the rail, and they increase with increase in speed. The lateral bending stresses in the inner rail on both curves almost without exception act to straighten the rail, and they decrease numerically with increase in speed. It will be seen that on the 10° curve at the speed corresponding to the superelevation the magnitude of the lateral bending stress in the outer rail under any wheel is almost the same as that in the inner rail under the wheel on the same axle. The action is to spread both rails at the wheels and to narrow the gauge slightly at a point between wheels. Some of the lateral bending stresses are relatively high. The lateral bending stress of 9 500 lb. per sq. in. in the outer rail of the 10° curve under the first wheel of the rear truck of the second car is 60% of the corresponding vertical bending stress and 80% of the vertical bending stress under the same wheel on straight track at 40 miles per hour. In general, the lateral bending stresses are nearly as high as those found with Locomotive 10254—the locomotive which developed the lowest lateral bending stresses.

It is worthy of note that for all the electric locomotives the lateral bending of the outer rail on both curves at the highest speeds is generally strongly outward under every wheel with a few minor exceptions, a condition that differs markedly from that found with the drivers of all the steam locomotives with which tests have been made.

14.—Lateral Bending Moments.—It is of interest to represent the lateral bending action in the rail by the lateral bending moments calculated from the observed lateral bending stresses by the use of the lateral section modulus, $\frac{I}{c}$, of the rail, and to plot the values under and between wheels to show the distribution and the intensity along the locomotive and cars. As the position, nature, and stiffness of the support of the rail laterally on the tie is not known, the values of the lateral pressures against the rail can not be estimated. It is evident that the conditions of the lateral support of the rail may be quite different from that of its vertical support by the tie and that the elastic properties of the lateral support are unknown. The test values give, therefore, the only information on the lateral action of the rail.

Figs. 40 to 43 give diagrams of the lateral bending moments in the outer and inner rails for all the locomotives and for the loaded freight cars. As before, positive bending is taken to be that which increases the curvature of the rail, and negative bending that which tends to straighten the rail. The graphical representation may be helpful in easily getting some idea of the action of the several wheels and the effect of speed on lateral bending of the rail, and also in making comparisons of the behavior of the locomotives. It should be borne in mind that the lateral bending moments in the rail may be taken to be expressive of pressures and movements trans-

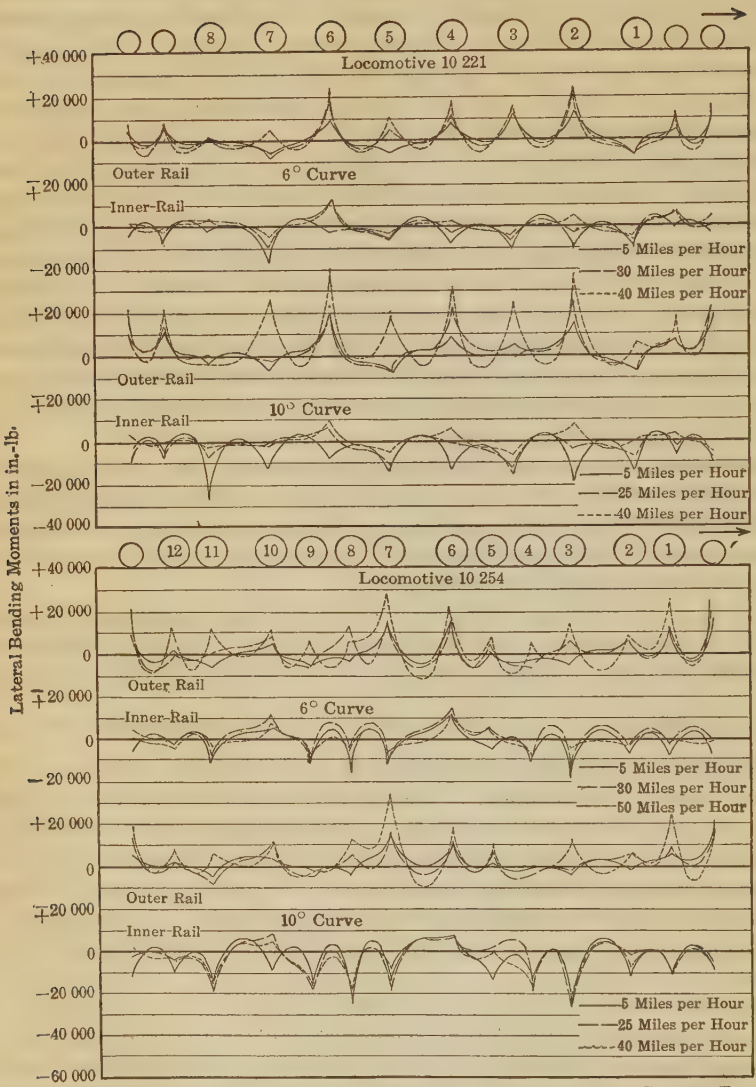


FIG. 40.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAIL OF CURVED TRACK WITH LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

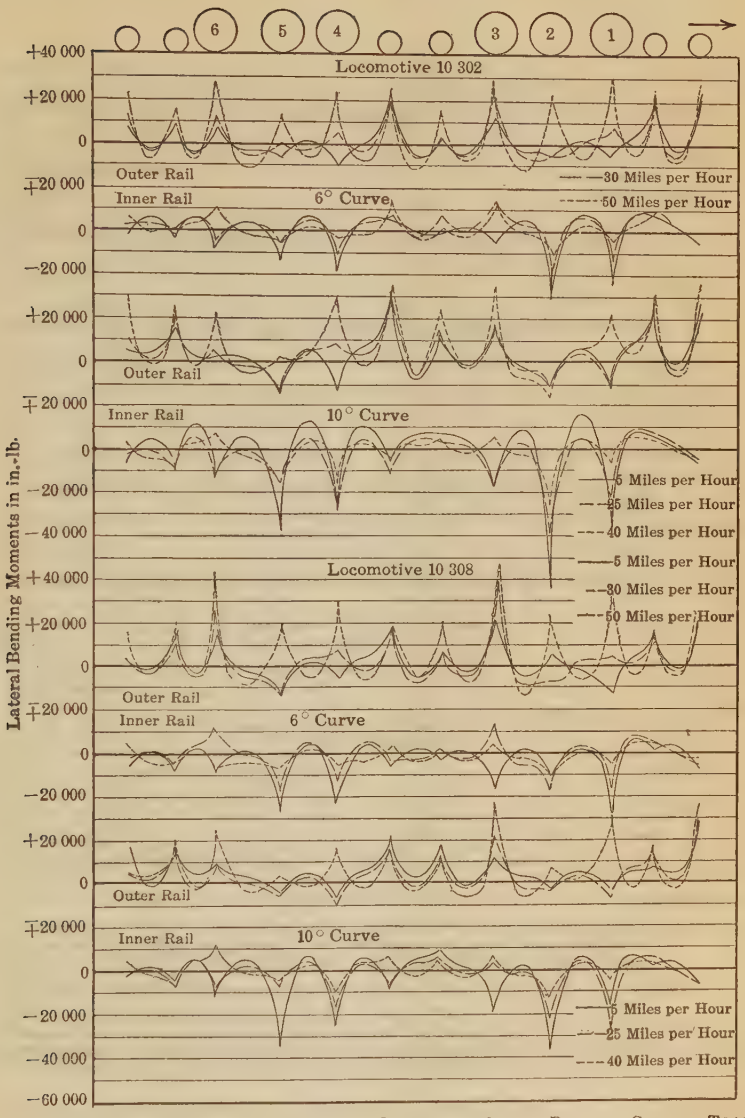


FIG. 41.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAIL OF CURVED TRACK WITH LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

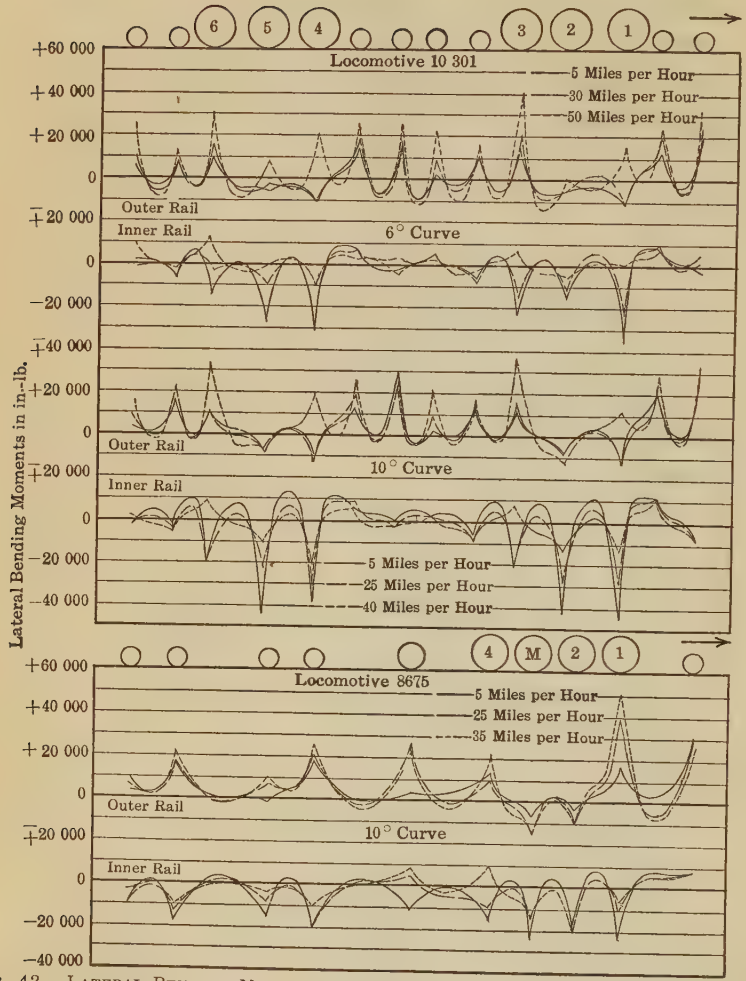


FIG. 42.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAIL OF CURVED TRACK WITH LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

mitted to ties and ballast affecting the maintenance of track line and surface. The relations between the lateral bending for the several locomotives have already been discussed under Article 13, "Lateral Bending Stresses in Rail on Curved Track"; the diagrams of lateral bending moments bring out clearly many of the characteristics of the locomotives there described.

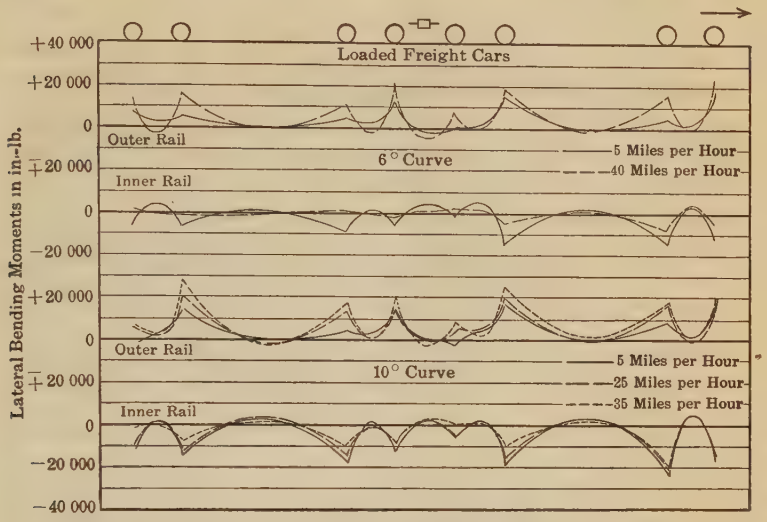


FIG. 43.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAIL OF CURVED TRACK WITH LOADED FREIGHT CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

The lateral bending moments for the outer and inner rails have been added algebraically and plotted for three locomotives in Fig. 44. The preceding diagrams (Figs. 40 to 43) brought out the large bending action developed in the two rails separately; these diagrams show the intense and severe lateral bending action produced in the track as a whole as the locomotive passes around a curve, tending to push the ties and ballast first to one side and then to another and to throw the curve out of line. In Fig. 45 the combined lateral bending moments for outer and inner rails show that for the loaded freight cars the lateral bending effect on the track as a whole is relatively small, since the moment in the two rails is of opposite sign. For the cars, then, the effect on the alignment is principally a spreading action and there is relatively little general distortion of the curve.

It will be noted that one type of locomotive produces effects on the track as a whole which in comparison to those produced on the two rails separately differ from the relative effects found with another type of locomotive.

15.—Lateral Movement of the Rails on Curved Track.—To obtain further information of conditions bearing on the maintenance of curved track, some measurements were made on the lateral movement of the head of the rail from its initial or original position as the locomotive passed around the curve at about 2 miles per hour. A bar and an attached Ames dial gauge were held between the rail and a fixed stake, 5 ft. away. The dial was read as

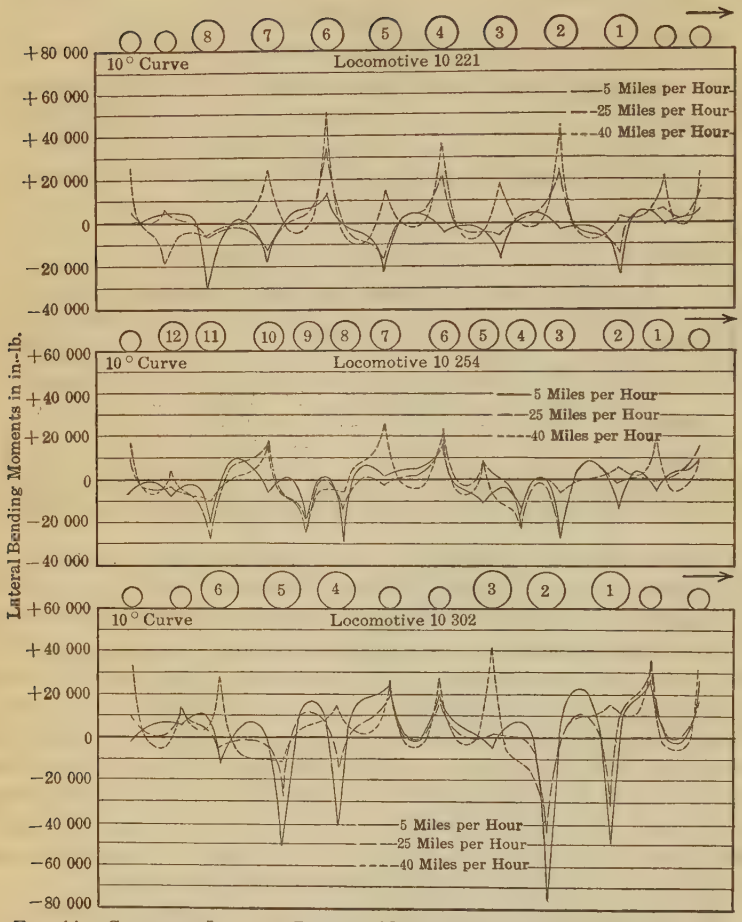


FIG. 44.—COMBINED LATERAL BENDING MOMENTS IN THE TWO RAILS OF THE 10° CURVE WITH LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

each wheel passed by. Any tilting of the rail will, of course, affect the lateral movement measured. Figs. 46 and 47 give some of the results of the tests. The values shown are generally averages of two tests.

With Locomotive 8675 (Mikado type) the outer rail of the 10° curve (see Fig. 46) is moved outwardly of the track more than 0.10 in. at the front truck wheel and the same distance at the fourth driver. The outward movement at the first wheel of each tender truck is also greater than at the second wheel. The inner rail is moved outwardly of the track at the first and second drivers about 0.22 in. This outward bending is continued by the other drivers, the trailer, and the wheels of the tender, ranging from 0.10 to 0.17 in. At all points the track is spread and the gauge widened beyond that of the unloaded track. The position taken by the two rails when the locomotive was backed over the curve (given by the dotted line) differs from that with the locomotives going forward principally in the greatly increased outward movement of the outer rail at the trailer (0.25 in.) (showing that the trailer guides the group of drivers, with apparent difficulty) and the increased outward movement of the inner rail at the second, third, and fourth drivers.

The movement of the rail on the 10° curve with Locomotive 10254 is peculiar (see Fig. 46). The outer rail is moved outwardly of the track by the first three wheels, inwardly by the first two drivers of the second group, and outwardly by the last two of this group and by the first one of the third group. The inner rail is moved outwardly throughout the length of the locomotive. The deflection is pronounced at the first driver of the second group and the second and third drivers of the third group, reaching 0.25 in. The widened gauge remains fairly constant for the whole distance. It is seen that the movement of the rails from their original position is in long swings as compared with the distances between the inflection points of the vertical and lateral bending stresses found in the rails themselves. The measurements taken when this locomotive was backed over the curve agreed very closely in magnitude and direction with the reversed position found with the locomotive going forward.

The outer rail of the 6° curve moved outward in the passage of Locomotive 10302, and the inner rail also for much of the length of the locomotive. Here, again, the increased gauge of the track remained fairly constant as the locomotive passed by. The record taken of the inner rail of the 10° curve shows a marked outward movement at the middle one of the first group of drivers—0.28 in.

The diagram given by Locomotive 10308 for the 10° curve shows an improvement over that given by Locomotive 10302. There is no large movement of either rail, and the gauge although widened remains nearly constant. Diagrams made with the locomotive backing are nearly identical with the reversed form of the forward ones.

The diagrams with Locomotive 10301 also are an improvement over those made by the original form of the locomotive. The widened gauge is nearly

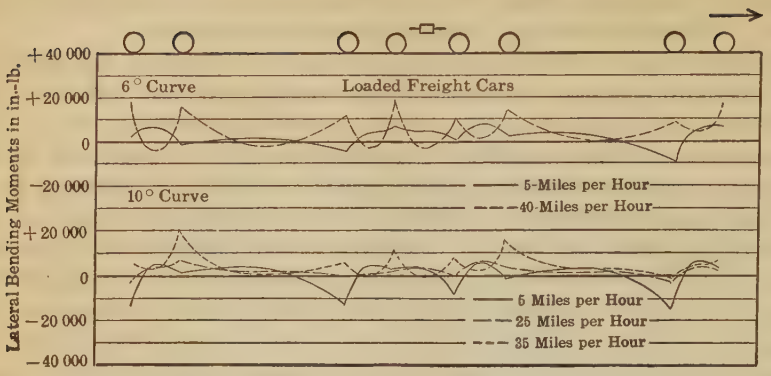


FIG. 45.—COMBINED LATERAL BENDING MOMENTS IN THE TWO RAILS OF THE 6° CURVE AND 10° CURVE WITH LOADED FREIGHT CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

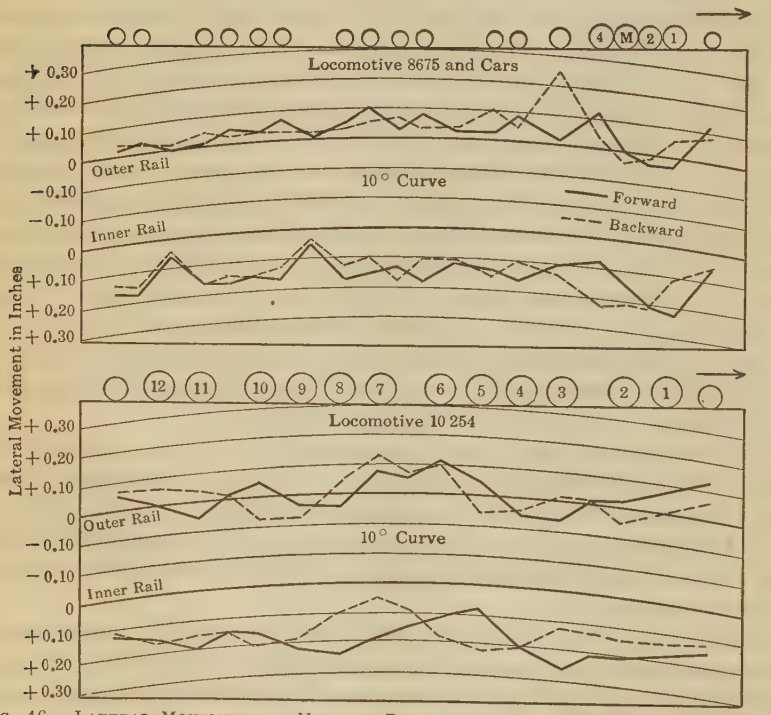


FIG. 46.—LATERAL MOVEMENT OF HEAD OF RAILS ON THE 10° CURVE AT SLOW SPEED. LOCOMOTIVES AND CARS OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

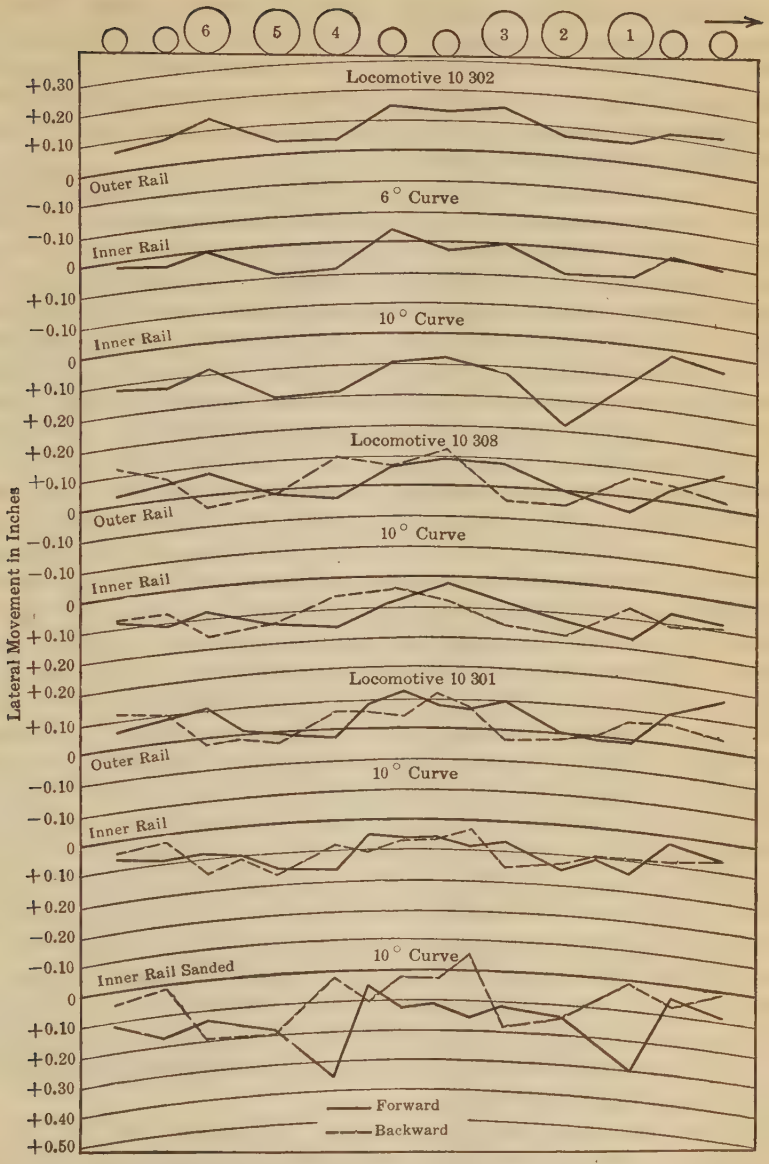


FIG. 47.—LATERAL MOVEMENT OF HEAD OF RAILS ON THE 6° AND 10° CURVES AT SLOW SPEED. LOCOMOTIVES OF THE CHICAGO, MILWAUKEE AND ST. PAUL RAILWAY.

constant. The movement of the rail with the locomotive backing was found to agree closely with that when it was going forward.

For the loaded freight cars (see Fig. 46), both outer and inner rails move outwardly, the latter in greater degree at this low speed. The movement of the outer rail is greater at the first wheel of a truck than at the second wheel. Both rails return nearly to their initial position at the middle of the length of the cars, a condition not found for the locomotive and tender. The spreading of the rails is greater than 0.2 in., the larger part of it being in the inner rail for the low speed.

It will be appreciated that the movement of the rails from their initial positions may not coincide with the magnitudes and character of the lateral bending stresses, since the latter may change quickly from point to point along the rail, while the former will be controlled by the combined action of a number of wheels. It will be noted that the distance between rails is always increased as the locomotive and cars go by. At higher speeds than that at which these observations were made, about 2 miles per hour, the movement of the rails may be expected to be similar to that described, except that the outer rail will move farther out and the inner rail not so far. Observations at higher speeds for the Eastern tests will be reported subsequently.

16.—Motoring and Regenerating.—It was thought possible that when a locomotive was motoring (pulling) the position of the wheels with respect to the rails on curved track would differ from that taken when the locomotive was operated with regenerative braking, and thus that the two methods of operation might give differences in wear of flanges on curves. The question was one of considerable interest to the railroad company. A possible source for such a difference lay in the direction of the friction between the drivers and the rail, the wheel tending to slip backward on the rail when the locomotive was motoring, and forward when it was regenerating. In traversing the curve the wheel on the outer rail must slip backward, or its companion on the inner rail forward, or possibly both. The wheel which slips longitudinally on the rail will thus be the one of the pair that has the best opportunity to slip laterally. It is seen then that the direction of possible longitudinal slip may determine conditions of lateral slip and thus also the position of the flanges with respect to the outer rail. As such changes would also modify the magnitude and direction of the lateral forces on the rail, it was thought well to make tests with two locomotives coupled together, one motoring and the other regenerating, and then reversing their operation and later their relative position. Any difference in the lateral bending of the rails might indicate differences in opportunities for flange wear during motoring and regenerating operation.

The tests were made on the 10° curve. The speed used was 25 miles per hour, approximately the speed corresponding to the superelevation of the track, and also about the greatest speed at which regenerative braking is used with so high a retarding effort. Values of the braking and motoring tractive force developed during the regeneration tests were read from the characteristic curves for the two types of motors by means of values of the line voltage

and field and armature current observed in the locomotive cab as the test section was passed. The characteristic curves were furnished by L. Wiley, Assistant Electrical Engineer of the Chicago, Milwaukee and St. Paul Railway. It is estimated that the draw-bar pull ranged from 35 000 to 55 000 lb. in the various runs. As operated, the speed was maintained practically constant.

In Figs. 48 to 51 are given the lateral bending stresses in the outer and inner rails for one locomotive motoring and another regenerating, the one motoring sometimes being in the rear. Locomotive 10304 is an original form of the Westinghouse-Baldwin locomotives and is similar to Locomotive 10302 which was not available at the time these tests were made.

A study of the lateral bending stresses given in the diagrams shows that the values are generally similar in magnitude and sign whether a locomotive is motoring or regenerating. There are small differences, but these are such as might be found in making tests from time to time. The values already given in tests with the locomotives coasting are also very similar in magnitude and sign. It may be concluded, then, that the lateral stresses in both outer and inner rail do not differ greatly whether the locomotive is coasting, motoring, or regenerating, at least at the speed corresponding to the superelevation of the track. It may be expected that the flanges of the wheels occupy approximately the same position with respect to the rail for the three conditions of operation.

Impressions of copper wire, taken with these tests showed that with Locomotive 10221 the flange of the first outer truck wheel bore against the outer rail, and also the first outer wheel of the rear truck, the second wheel of each of these trucks being also close to the rail. This condition is true whether the locomotive was motoring or regenerating and whether it was in front of or behind another locomotive. It was also found that the flanges of the second inner driver of each group were usually close to the inner rail, even though the lateral bending stresses in the outer rail at the corresponding outer wheel indicated a large outward bending. From these impressions and the lateral bending stresses it is not clear how any but the first group of drivers is guided around the curve. It is evident that groups of two axles have an advantage in traversing curves.

Impressions of copper wire taken with Locomotive 10308 showed that the flange of the first outer truck wheel bore against the outer rail, as did the outer truck wheel just ahead of the second group of drivers and the first outer wheel of the rear truck, whether the locomotive was motoring or regenerating, and the same condition was found when the locomotive was coasting over the curve. It is evident that the flanges of the two wheels named furnish the guiding action for the two groups of drivers. The same conditions were found with Locomotive 10304, but it will be noted that the lateral bending stress at the outer truck wheel ahead of the second group of drivers was greater than with Locomotive 10308, as was true in the test with Locomotive 10302 when coasting. In the tests with all the Westinghouse-Baldwin locomotives, at the speed of 25 miles per hour, usually the flange of the rear inner

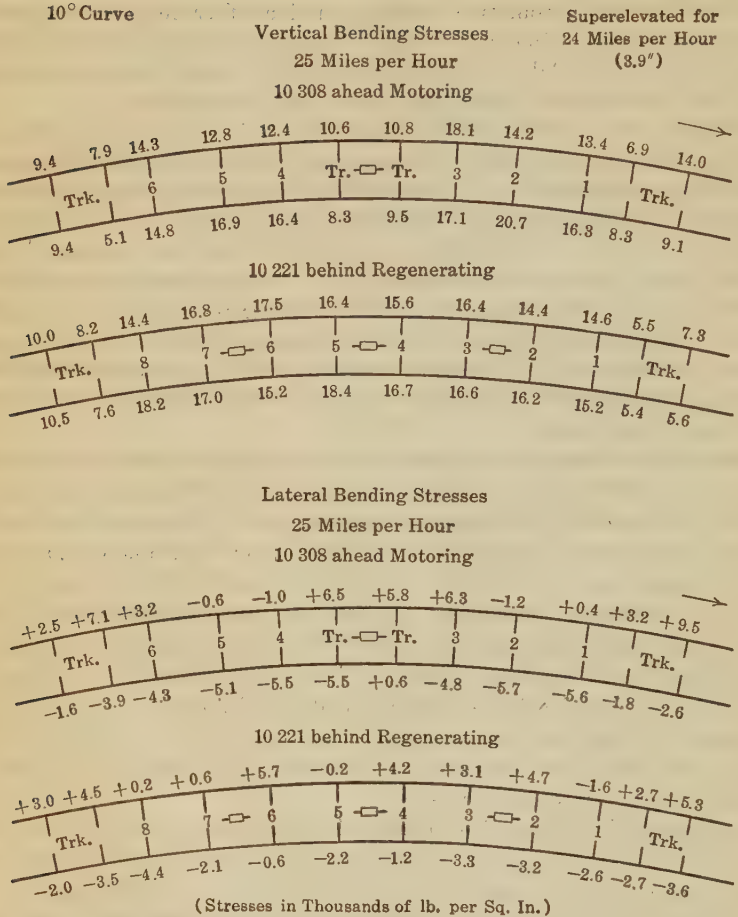


FIG. 48.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE. LOCOMOTIVE 10308 AHEAD MOTORING; LOCOMOTIVE 10221 BEHIND REGENERATING.

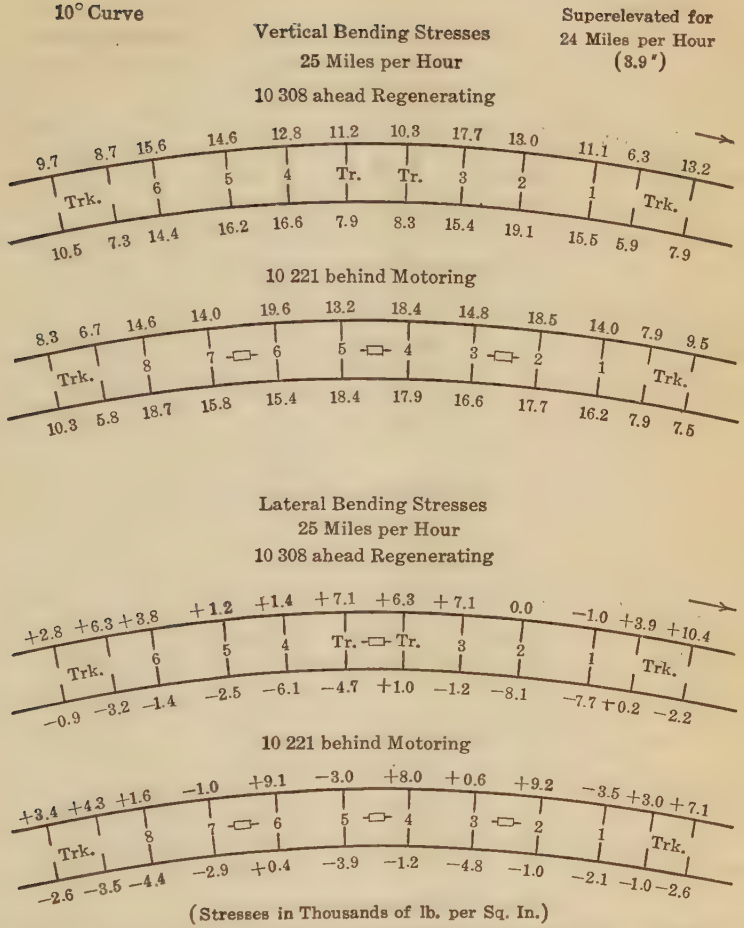


FIG. 49.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE. LOCOMOTIVE 10308 AHEAD REGENERATING; LOCOMOTIVE 10221 BEHIND MOTORING.

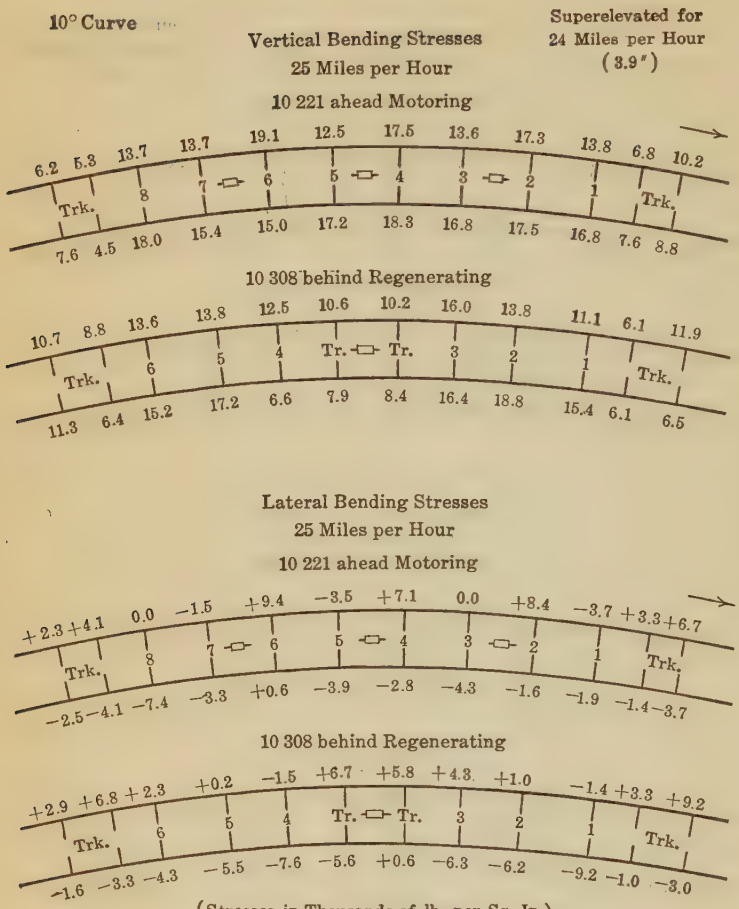


FIG. 50.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE. LOCOMOTIVE 10221 AHEAD MOTORING; LOCOMOTIVE 10308 BEHIND REGENERATING.

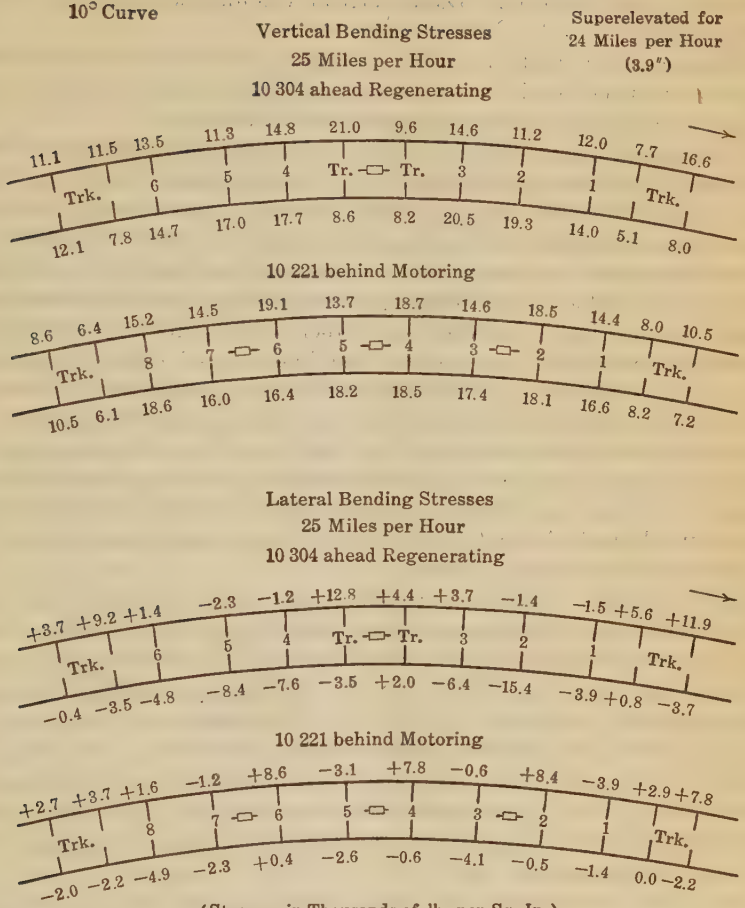


FIG. 51.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE. LOCOMOTIVE 10304 AHEAD REGENERATING; LOCOMOTIVE 10221 BEHIND MOTORING.

driver of each group, and, in some cases, those of both the second and the rear driver, were close to the inner rail, while at the same time the companion driver produced outward bending in the outer rail.

It was also found that the vertical bending stresses developed in the motoring-regenerating tests (see Figs. 48 to 51) did not differ greatly with the two methods of operating and were much the same as those found with the same locomotives when coasting. These vertical bending stresses are, therefore, not discussed.

It is thought also that the lateral bending stresses for a locomotive motoring or regenerating would not be relatively different at speeds higher than the speed corresponding to the superelevation of the track.

17.—General Discussion of the Tests on the Chicago, Milwaukee and St. Paul Railway.—It may be of advantage to review some of the facts brought out in the preceding discussion, to make comparisons, and to call attention to relations of various kinds that are apparent from the results of the tests.

The mean stress in base of rail on straight track at the low speed had much the same value as that found by calculation by the analytical method using the nominal loads, the differences rather emphasizing the variations from the nominal loads that may occur. The influence of the close spacing of wheels in decreasing the stress in rail is well illustrated by Locomotives 10254 and 10221, the stress under a driver of the former being only about 50% of that which would be developed if the wheels were very far apart while that under the drivers of the latter was about 85% of a corresponding value. The average of the mean stresses in the outer and inner rails for all the wheels on the curved track did not differ much from the calculated values, being somewhat greater on the 10° curve.

The mean stress in base of the 90-lb. rail under the drivers on straight track at 5 miles per hour ranged from 8 000 lb. per sq. in. for the General Electric passenger locomotive and 12 000 for the Mikado type (not including counterbalance effect) to 15 500 for the Westinghouse-Baldwin locomotive (with an average stress for one driver of 19 700 lb. per sq. in.) and 17 000 lb. per sq. in. for the General Electric freight locomotive. The mean stress in the rail at the same speed under the wheels of the loaded freight cars with a wheel load of 18 500 lb. averaged 11 000 lb. per sq. in. There was, of course, a belt of values with a range above and below the average value reported for any wheel of locomotive or cars.

The increase in stress in rail with increase in speed was relatively small for all the locomotives. It may be remarked that variations may be expected in the values of the ratios, as there are many variable conditions in such tests, this being especially true when the measured stresses are small; thus, for a stress of 8 000 lb. per sq. in., an increase of 10% is only 800 lb. per sq. in., and changing conditions may easily make differences nearly as great as this. For the passenger locomotives (except for Locomotive 10302) the increase in stress on straight track from 5 to 60 miles per hour may be said to be about 12% (see Fig. 14). The corresponding increase in stress was about 1 500 lb. per sq. in. The speed effect from 5 to 40 miles per hour was relatively smaller.

That of the General Electric freight locomotive was not large. That of the Mikado locomotive was larger, but was only 10%; this does not include the effect of counterbalance. On the 6° curve the speed effect differed but little from the value for straight track. The 10° curve gave a somewhat higher value, but the difference is not marked. The values for the loaded freight cars on straight track and 10° curve are greater, but for these the increase in mean stress is only 13% and 25%, respectively. All these values of the effect of speed are smaller than were obtained in former tests. It should be borne in mind that the values noted are all average values; in all the locomotives and cars there are increases under individual wheels that are much larger than the average given.

The relation of the speed effect for dead or unsprung load to that carried on effective springs is of interest, as is also the relation of diameter of wheel to speed effect. It is frequently stated that the effect of speed on track is inversely proportional to the square of the diameter of the wheel and directly proportional to the dead weight carried. Sometimes, the first power of the diameter is given instead of the square. Table 1 gives the unsprung weight carried by one driver of the General Electric freight locomotive as 8 100 lb., equal to 29% of the average total load on the driver, the corresponding values for the General Electric passenger locomotive being 4 800 lb. and 24%, and for the Westinghouse-Baldwin passenger locomotive, 3 900 lb. and 12 per cent. The diameters of the drivers of the three types of locomotive are 52, 44, and 68 in., respectively. In Table 5 and Fig. 14 are given the average values of the speed effects found, as measured by the mean stress in base of rail. A study of these values and of the other data of the tests* does not indicate that the effect of speed on bending stresses in rail varies with diameter of wheel and proportion of unsprung weight to the extent given by the principles quoted previously. The variations in the values due to a number of causes do not permit the effect of diameter of wheel and amount of unsprung weight to be segregated, but it is evident that their effects, although important, are far smaller than would result if the principle cited were sound. The effect of diameter of wheel, within the limits of size and speed considered, from analytical considerations would not seem to be very great for good track, whatever might be the effect on the bearing strains in the rail (detrusion) or in the resistance to traction—the latter being relatively small. The assumption that the speed effect varies with the proportion of unsprung load implies that the sprung load has a negligible effect, thus overlooking the defects of the action of the springs and equalizing systems and the flexibility of the track itself. Doubtless, too, a load applied between the wheels of a pair exerts less speed effect on one rail than if half of it were applied to the one wheel. Poorly designed and maintained equalization systems and inadequate springs, too, may detract from the usefulness of springs. It is not intended by these remarks to belittle the need for providing adequate springs for a large proportion of the load. It may be added that the condition of the track surface

* In Locomotive 8675 the unsprung weights for the first, second, third, and fourth drivers are 20, 22, 33, and 20% of the wheel loads, respectively. The diameter of the drivers is 63 in. The increase in stress in rail under the drivers with change from 5 to 40 miles per hour on straight track is about 9%, not counting the effect of counterbalance.

must have an important influence on the speed effect; uneven track, that is, track with a succession of hard spots and soft spots may be expected to give large speed effects. Attention should also be called to the advantage of having adequate easement curves when the proportion of unsprung weight is large, particularly when this weight is low, as when it is carried directly on the axle.

The foregoing discussion relates to the average values of the mean stress in base of rail. Due to the lateral bending of the rail, the ratio of stress at outside edge to mean stress will vary. On straight track (see Tables 6 and 7), the average of these ratios for the various wheels of the several locomotives ranged from 0.82 to 1.33, the first value being under the trailer of the Mikado locomotive and the second under a driver of a Westinghouse-Baldwin passenger locomotive, the lateral bending stress in the second instance having a value of 4 500 lb. per sq. in. The ratios were generally different for the two sides of the locomotive, sometimes markedly different, and the ratio at one wheel differed from that at another. Generally, there were one or more wheels in each locomotive that gave a markedly higher outward lateral bending of the rail than the others, the reason for the higher stresses not being apparent.

Also, for a given wheel, the individual observations of lateral bending stresses in the rail of straight track varied widely from the average stress corresponding to the average ratio just considered. Lateral bending stresses as great as 9 000 lb. per sq. in. were found in connection with a mean stress or vertical bending stress of 22 000 lb. per sq. in., making a stress of 31 000 lb. per sq. in. at the outside edge, though there were few of this magnitude. It is evident, of course, that these lateral stresses and the accompanying lateral movement of the rail are factors which produce stresses and movements in ties and ballast and thus affect track maintenance and also enter into the maintenance requirements of the locomotives. It may be said, however, that the lateral bending stresses and lateral movements found in the tests with these locomotives are, in general, less than have been found in some of the tests of straight track made previously. With the electric locomotives the absence of stress due to counterbalance was especially noticeable.

It is to be expected that with the great diversity in wheel loads, wheel spacing, wheel grouping, and articulation of running gears and devices for regulating flexibility, the four types of locomotives in traversing the curved track will show a variety in the magnitude and distribution of the lateral bending stresses in rail, in the division among the wheels of the work of guiding the locomotive around the curve, and in effecting the lateral slip of the wheels on the rail which is an essential element in negotiating a curve. Differences may be expected in the flexibility of the locomotives, both as a whole and in their various parts, and also in the effect on the alignment of the track and its necessary maintenance. A discussion of the effect of the details of design of the different locomotives and their influence on the track as shown by the tests cannot well be taken up at this time. The mechanical departments of the railroads and the builders of locomotives have long given attention to the requirements of locomotives for traversing curved track. The results herein

reported may be expected to be of service in the further improvement of details of this part of locomotive design. Some of the methods and results may be suggestive of ways for securing information bearing on the effect and the usefulness of devices and of methods of correcting defects which may be found to exist.* The data reported here will give opportunity for the study of various questions which have been raised concerning certain features of locomotive design. It is to be expected that conditions giving unusual or unexpected stresses in the rail will produce similar effects in the locomotive structure.

The tests show that the lateral bending stresses in the rail on curved track, and, therefore, the lateral pressures on the rail, are relatively very great. For the curves used (6° and 10°) relatively large lateral pressures on the rail must be accepted as a necessity; the problem is how by proper design and maintenance to keep the values as small as possible and to make the track structure as resistant as is feasible.

All the locomotives behaved well in traversing the curved track even at the highest speed run, although, of course, differences were noticeable. The speeds of 50 miles per hour on the 6° curve and 40 miles per hour on the 10° curve run by the passenger locomotives were rather high, in view of the superelevation used, but the locomotives ran the curves very smoothly except where the alignment was not good. The speed corresponding to the superelevation of the track was only three-fifths of the maximum speeds run, but the tests do not indicate that the maximum lateral bending stresses were unduly increased by reason of this difference.

The effect of imperfect alignment in parts of the curved track on the behavior of the locomotives, as in the vicinity of the pile bridge on the 10° curve, and also the effect of an easement curve that was imperfectly maintained serve to emphasize the importance of maintaining excellence in line and surface if trains are to be operated at high speeds around such curves. At the higher speeds it appears also that easement curves of adequate length are very useful in adjusting the tracks and coupling connections to take the curve under the best conditions and thus to reduce the wear of both locomotive and track.

The tests have again brought out the inter-relation of quality of maintenance of both track and rolling stock to effects of traffic on the maintenance of both and thus emphasize the value of keeping up a high standard of excellence in the maintenance of track and of locomotives and cars. High excellence in the make-up of the track structure and in the design of locomotives and cars likewise contributes to low maintenance costs.

* An excellent illustration of improvements made by changes in the design of locomotives after track tests had been made, whereby a critical excessive stress developed in the rail of a 10° curve was decreased by one-third, is given in *Circular No. D. V.-344* of the Mechanical Division of the American Railway Association, a paper on "The Relation of Track Stresses to Locomotive Design", by C. T. Ripley, Chief Mechanical Engineer of the Atchison, Topeka and Santa Fé Railway. The paper brings out in an excellent way the intimate relation between locomotive design and stresses in track.

III.—Tests on Eastern Railroads

18.—The Phenomena.—The tests on Eastern railroads were undertaken mainly with the view of obtaining information on the effect of canting the rail inwardly of the track, upon the stresses in the rail, and upon other matters affecting track maintenance, as compared with the results found with upright rail. The tests also gave information on the action of track laid with the heavy rail used on three of the railroads.

Whether there are advantages in so supporting the rail that its axis is inclined inwardly of the track instead of being vertical has been a mooted question. The advocates of canting have held that by adzing the ties to give the right slope or by using tie-plates with the outer end thicker than the inner one (called inclined tie-plates), the center of the bearing of the tread of the wheels of locomotives and cars will be more nearly on the center of the head of the rail and the wear of the rail will extend over a greater width of the head, thus increasing the life of the rail, and that the load will be more directly transmitted to the tie than is the case when the axis of the rail is vertical. It is further said that canting tends to prevent widening of the gauge under traffic and otherwise reduces the cost of maintenance. The inclined tie-plate also usually projects farther on one side of the rail than on the other, the greater thickness at the side having the greater projection may present an economic advantage in design. The advocates of a vertical position for the rail feel that better results will be obtained with a flat tie-plate proportioned so that it projects outwardly from the rail a distance sufficiently greater than its inward projection that the resultant of the bearing force of wheel on rail will pass through the center of the tie-plates; that the condition of the tread of wheels is so diverse and the conditions of track and rolling stock affecting the direction and magnitudes of the lateral pressures so variable that it is impracticable so to cant the rail as to give the advantages claimed; that it is difficult to provide a construction near and through turnouts that will be substantial and workmanlike without undue expense; and that dissatisfaction with vertical rail may generally be laid to neglected maintenance and to lack of tie-plates or to small and poorly proportioned tie-plates.

On curved track the effect of canting the rail may be expected to be more complicated than on straight track, and the conditions may differ for inner and outer rail and under the several wheels of the locomotive and cars. It is evident then that tests and observations will be needed on both straight and curved track and that separate consideration should be given to the two kinds of track.

As the test party had had no previous experience with the problem, all the tests had to be made in the ways that seemed to give the best promise of results, and the utility of the methods could not be determined until after the data were reduced and assembled. It had been judged that the lateral bending stresses in the rail, as measured by the stremmatograph, would furnish the most useful information obtainable and that measurements of the lateral movement of the rail and of its tilting as the load passed by would serve well as an

auxiliary means of obtaining information. The lateral bending stresses were found to give information of value bearing on the canting of the rail and other special conditions of the track, but it is now seen that a much larger number of measurements of lateral movement and lateral tilting of the rail at several points on the track with a better and more systematic development of methods would have furnished information more directly applicable to the problems than was appreciated in advance.

It is apparent that the phenomena of lateral movement, lateral tilting, and lateral bending of the rail are intimately connected not only with the condition of the tread of the wheel and the position of the rail with respect to the vertical, but also with the amount of eccentricity of the tie-plate, the nature of the wood of the tie and its elastic properties, the condition of the ballast support, and the lateral movements and forces of the locomotive and cars. These elements are bound together in an intricate way. To find the influence of a single one would be a difficult task. To help to understand the action of these factors a further statement of the conditions and factors present when wheel loads pass over the track may be useful. Some of them may seem minor matters and others indefinite—all of them will be subject to considerable variation. The conditions to be found on straight track will be taken up first:

(*a*) The gravity load transmitted by wheels to rails on track that is level transversely constitutes a vertical force, which, of course, has no horizontal or lateral component.

(*b*) Lateral movements of the load due to variations in track and oscillations and nosings of locomotive and cars produce lateral forces acting against the rail, either inwardly or outwardly of the track. These forces will be applied to the rail by friction between the tread of the wheel and the head of the rail, and less frequently by pressure from the flange of the wheel itself.

(*c*) Lateral changes in the direction of motion of the wheels of a frame or truck from time to time, whether by lateral slipping or otherwise, will develop lateral forces on the rail that are of consequence in contributing to the variability of results.

(*d*) It has been shown in previous progress reports that the ordinary coned wheel running along an upright rail develops a lateral bending in the rail. For perfect conditions of wheel, rail, and tie, it would appear that this outward thrust may be connected with the differences in distances traveled by the wheel tread at the two edges of the surface of contact, due to the differences in the circumferences of the wheel at these two points and also possibly to an unsymmetrical area of contact of wheel on rail when the center of bearing is at one side of the middle of the head. This lateral bending of the rail persists even after the wheels have come to rest. It would be of interest to learn whether rail that is so canted as to bring the center of bearing at the middle of the head would develop lateral bending of the rail to the same extent.

(*e*) The resultant of the vertical load and the lateral force may not pass through the mid-point of the base of the rail. The use of an eccentric

tie-plate, that is, one having a greater projection on one side of the rail than on the other, is, of course, based on the belief that the resultant is not vertical, and, since the longer projection is placed on the outside, that the resultant inclines outwardly. As the position of the resultant will be variable, the design of the tie-plate should be based upon the average position of this force.

(*f*) For other positions of the resultant the distribution of pressure on the tie will not be uniform from end to end of the plate. If e is the relative eccentricity of the resultant of the applied forces, given in terms of the length of the tie-plate (for example, $\frac{1}{10}$ when the resultant passes 1 in. from the middle of a tie-plate 10 in. long), and p_0 is the average pressure of tie-plate on the tie, it is easy to show that the intensity of pressure, p, at an end of the tie-plate will be given by the equation:

$$p = (1 \pm 6\,e)\,p_0 \qquad (34)^*$$

when the relative eccentricity is less than $\frac{1}{6}$ and,

$$p = \frac{2}{3}\,\frac{p_0}{\frac{1}{2} - e} \qquad (35)$$

at the outside end when the relative eccentricity is greater than $\frac{1}{6}$. In Fig. 52 three distributions of the bearing pressure are shown. For uniform pressure the resultant of the applied forces will pass through the mid-point of the plate. If the resultant of the applied forces cuts the bottom of the plate one-sixth of its length away from its mid-point $\left(\text{relative eccentricity, } \frac{1}{6}\right)$, the pressure at one edge will be double the average and that at the other will be zero. If the resultant passes through a point one-fourth of the length from the mid-point, the pressure at one edge will be two and two-thirds times that when uniformly distributed over the tie-plate, and zero at and beyond the quarter-point of the plate. In the latter case the resultant would pass near or outside the edge of the base of rail. In this connection a distinction should be made between the eccentricity of the resultant of the applied forces with respect to the tie-plate and the eccentricity of an unsymmetrical tie-plate, the latter being used as the distance between the mid-point of the length of the tie-plate and the axis of the rail as laid.

(*g*) When the resultant passes to one side of the mid-point of the tie-plate, the tie is unevenly compressed along the length of the plate. As the modulus of elasticity of wood for bearing pressure is relatively small (less than one-fiftieth part of that for tension and compression lengthwise of the fibers, counting the effect of both detrusion and compression), there may be a relatively large difference in the depth that the plate depresses the tie at the two

* This equation number follows the last number used in the third progress report, *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 980.

ends of the plate. As a result, the rail will tilt under the load, the head of the rail will move inward or outward, and the gauge will narrow or widen. Besides, if the intensity of bearing pressure is greater than the elastic bearing limit of the wood, repetitions of such applications will result in cutting the tie and give permanent changes in gauge and alignment. It is evident that the length of tie-plate and its eccentricity will have an important influence upon the action of the rail and tie under load and thus, also, upon matters of track maintenance. This tilting and lateral movement may be quite independent of the lateral bending of the rail due to the lateral forces already referred to.

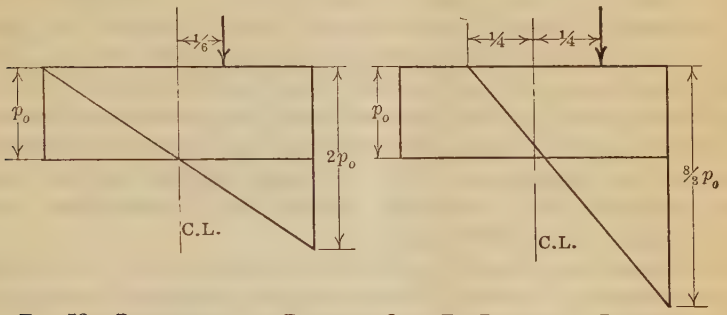


FIG. 52.—DISTRIBUTION OF PRESSURE OVER TIE-PLATES FOR DIFFERENT POSITIONS OF RESULTANT.

(*h*) The flexural position of the tie at the rail may also affect the inclination of the rail, although not to the same extent as the tie compression. If the tie at this point slopes outwardly when under load, as in the case of a centerbound tie, the rail will tilt outwardly; if the slope is inward, as with an endbound tie, the rail will tilt inwardly.

(*i*) The order of magnitude of the depressions, slopes, tilting, and lateral movement may not be large, and yet the effect on track maintenance may be of importance.

(*j*) It is apparent that with all the variations of locomotives and cars and of track conditions great variations will be found in the effect on rail and tie. One rail may differ considerably from the other, and the conditions will vary from point to point along the track. Both average results and ranges in values should be considered and a study made of the meaning of the variations.

On curved track the conditions are further complicated. The transverse inclination of the track gives inclination to both rails, changing the inclination of the line joining the middle of the head of the rail and the middle of the length of the tie-plate, in effect increasing the eccentricity of the tie-plate of the outer rail for speeds below the speed corresponding to the superelevation and decreasing it for speeds above it, and having the opposite effect for the inner rail. As was shown in the third progress report, the wheels of a fixed frame must be made to change direction in traversing the curve; the flange of the leading outer wheel, or of a wheel ahead of the group, by bearing against the outer rail acts to cause lateral slippage of two or more of the leading wheels

of the frame and thus develops an important lateral force outwardly on the rail. By the turning action other outward or inward lateral forces in both rails are developed, which cause important lateral bending stresses in the rail. These various lateral forces and the combined effect of centrifugal force and superelevation may be expected to influence the effect of canting the rail and of eccentricity of tie-plate.

19.—The Track.—The tests were made on track in the condition found at the time. No special work had been done before the test except that on the Baltimore and Ohio Railroad inclined tie-plates had been replaced ten days before on a stretch of straight track and on a 7° curve in order that tests might be made with flat tie-plates comparable to the conditions of track having inclined tie-plates. The stretches of track chosen for the tests appeared to be as uniform as the general run of the adjacent track. They are thought to be representative of the track found on the railroads in the vicinity of the tests except possibly the track with flat tie-plates on the tangent and 2½° curve of the Richmond, Fredericksburg and Potomac Railroad which will be described subsequently. For all except the two locations just referred to, the track was in excellent condition and representative of high-grade track under heavy traffic service.

Table 12 gives data of rail, tie-plates, ties, ballast, gauge, and superelevation of outer rail on curves. Table 13 gives properties of the sections of the rails where tests were made and Fig. 53 the rail sections. Fig. 54 shows the design of the tie-plates. All inclined tie-plates had a slope of 1 in 20.

The average cant of the rails at the test locations, both straight track and curved track, is given in Table 14. The cant was found by placing a straight bar across the track under the rails and measuring the deviation of the base of rail from this bar, no load being on the track. The value thus found varied from point to point along the track, regardless of the kind of tie-plate used and the condition of the track, and the values in Table 14 are the averages of five observations on each rail. In several cases, the rail was found to be canted outwardly of the track, one such instance being on straight track.

The tests on the Baltimore and Ohio Railroad were conducted on the eastbound track at Harpers Ferry, W. Va. On the straight track the rail was worn only slightly. On the two 7° curves the rail was in good condition, except that on the 7° curve having flat tie-plates, the head of the inner rail had been mashed down considerably, this condition existing before the tie-plates were changed. The alignment and surface were good. The two curves had a high superelevation (6.7 and 7.1 in.), and the greatest speed used, 40 miles per hour, was only slightly above that corresponding to the speed of superelevation. With the heavy traffic over this track, the tests were conducted with difficulty.

The tests on the railroad of the Reading Company (formerly the Philadelphia and Reading Railway) were conducted on eastbound track at and near Myerstown, Pa. From Myerstown west there are four tracks and a few

TABLE 12.—DATA OF THE TRACK.

Track.	Tie-plates.	Rail section.	Year of rolling.	Ties, in feet and inches.	Number of ties, in 33 ft.	Depth of ballast, in inches.	Gauge, in feet and inches.	Superelevation, in inches.	Speed corresponding to superelevation, in miles per hour.
Baltimore & Ohio R. R.:									
Straight	Flat	130-lb. R.E.	1921	Treated oak, 7 by 9 by 8 ft. 6 in.	18	18, Rock	4 ft. 8 ¾ in.	...	..
Straight	Inclined	"	"	"	"	"	4 " 8 9/16 "	...	..
7° curve	Flat	"	"	"	"	"	4 " 8 5/16 "	6.7	38
7° curve	Inclined	"	"	"	"	"	4 " 9 1/16 "	7.1	39
Reading Co.:									
Straight	Flat	"	1923	Treated oak, 7 by 9 by 8 ft. 6 in.	18	12, Rock	4 " 8 7/16 "	...	..
Straight	Inclined	"	"	"	"	"	4 " 8 5/8 "	...	..
Lehigh Valley R. R.:									
Straight	Inclined	136-lb. L. V.	1916	Treated oak, 7 by 9 by 8 ft. 6 in.	20	18, Rock	4 " 8 9/16 "	...	..
10° curve	Inclined	"	1923	"	"	"	4 " 8 5/8 "	6¼	31
Richmond, Fredericksburg & Potomac R. R.:									
Straight	Flat	100-lb. Am. Soc. C. E.	1914	Oak, 7 by 9 by 8 ft. 6 in.	19	Gravel	4 " 8 9/16 "	...	..
Straight	Inclined	100-lb. R. E.	1922	"	18	"	4 " 8 5/16 "	...	..
2⅛° curve	Flat	100-lb. Am. Soc. C. E.	1914	"	21	"	4 " 8 11/16 "	4.0	49
5° curve	Inclined	100-lb. R. E.	1921	"	18	"	4 " 9 ⅛ "	5.4	40

miles eastward there are three, so that the traffic on the test stretch is extremely heavy. The rail had been laid two months before. A few of the inclined tie-plates were loose. The ties were large and in good condition. The track was in general good condition.

TABLE 13.—PROPERTIES OF SECTIONS OF RAILS ON TEST TRACK.

Rail section (full section).	Area, in square inches.	Moment of Inertia.		Section Modulus.		
		For horizontal axis.	For vertical axis.	Horizontal Axis.		Vertical Axis.
				Base.	Head.	Base.
90-lb. A. R. A.-A.....	8.80	38.7	7.5	15.2	12.6	2.9
100-lb. Am. Soc. C. E...	9.84	44.0	9.8	16.1	14.5	3.4
100-lb. R. E.....	9.95	49.0	9.4	17.8	15.1	3.5
130-lb. R. E.....	12.71	77.4	14.5	25.6	20.8	4.8
136-lb. L. V.....	13.35	86.6	17.7	28.3	22.0	5.4

The tests on the Lehigh Valley Railroad were conducted on eastbound track, 6 miles west of Mauch Chunk, Pa. The traffic was heavy. The tests were made on straight track and 10° curve (the latter being called the Ox Bow Curve), both having inclined tie-plates. The rail was in good condition, though that on the 10° curve, as shown in Fig. 101, was considerably worn even though the outer rail had been laid only two months before, the inner rail having been taken from the outer rail at the same time where it had been in service about six months. The tie-plates had been in the track and on the same ties since 1916. The surface and alignment were good, and the ballast was unusually good. On the 10° curve the superelevation (6¼ in.) corresponded to a speed of 31 miles per hour, and a speed of 35 miles per hour was used in the tests.

The tests on the 5° curve on the Richmond, Fredericksburg and Potomac Railroad were made north of Cherry Hill, Va., on track with inclined tie-plates, and near Brooke, Va., on straight track laid with flat and with inclined tie-plates and on a 2½° curve with flat tie-plates. The 5° curve was in excellent condition, except that the alignment was only fair. The superelevation was 5.4 in., corresponding to a speed of 40 miles per hour. The straight track having canted tie-plates was in excellent condition. The track having flat tie-plates, both the straight track and the 2½° curve, was not in as good condition; it was planned soon to relay these stretches with new rail and inclined tie-plates. At the test location of straight track with flat tie-plates, near the beginning of the 2½° curve, the right rail was 2 in. lower than the left, the location being chosen as the only straight track with flat tie-plates available. All tests were made on southbound track.

Measurements of track depression were not made except in an incomplete manner, but it is thought that the modulus of elasticity of rail support, u, was about 2 000 on the Baltimore and Ohio Railroad, the Reading Company, and the Lehigh Valley Railroad. The track structure was apparently both solid and stiff.

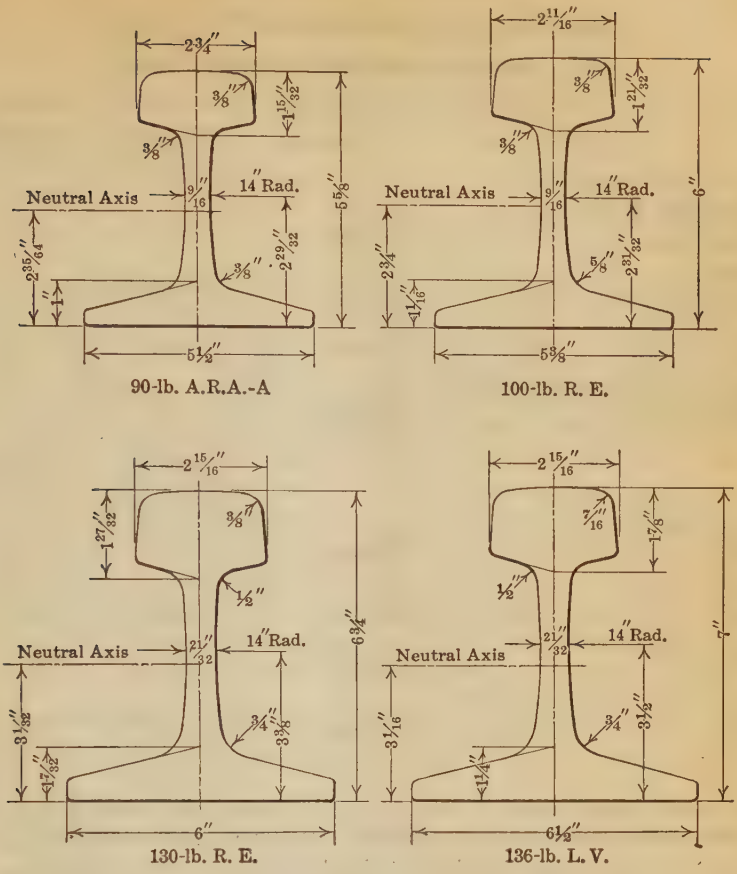


FIG. 53.—SECTIONS OF THE RAILS.

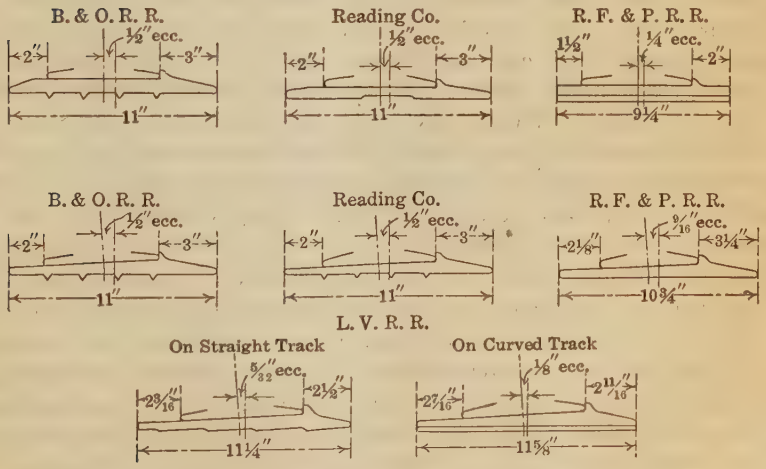


FIG. 54.—SECTIONS OF THE TIE-PLATES.

TABLE 14.—Cant of Rails on Test Track.

(The minus sign indicates an outward cant.)

Location.	Tie-plate.	Average cant of left rail.	Average cant of right rail.
Baltimore & Ohio R. R.:			
Straight track	Flat	1:67	1:54
Straight track	Inclined	1:15	1:17
Reading Co.:			
Straight track	Flat	1:100	00
Straight track	Inclined	1:17	1:16
Lehigh Valley R. R.:			
Straight track	Inclined	1:18	1:18
Richmond, Fredericksburg & Potomac R. R.:			
Straight track	Flat	00	—1:30
Straight track	Inclined	1:21	1:18
		Average cant of outer rail.	Average cant of inner rail.
Baltimore & Ohio R. R.:			
7° Curve	Flat	—1:120	—1:120
7° Curve	Inclined	1:23	1:23
Reading Co.:			
1° Curve	Flat	1:60	—1:30
1° Curve	Inclined	1:15	1:24
Lehigh Valley R. R.:			
10° Curve	Inclined	1:36	1:27
Richmond, Fredericksburg & Potomac R. R.:			
2½° Curve	Flat	—1:115	1:32
5° Curve	Inclined	1:26	1:42

20.—The Locomotives and Cars.—In Fig. 55 are given the diagrams for the locomotives used in the tests on the four railroads—wheel loads and spacings, diameter of drivers, and number of the locomotive. The locomotives were used as they were found in service. The locomotive of the Baltimore and Ohio Railroad had been in service for only a short time, that of the Reading Company was relatively new, and that of the Lehigh Valley Railroad had been in the shop shortly before. The locomotive of the Richmond, Fredericksburg and Potomac Railroad was older, but was in fair condition. The Pacific type locomotive is the standard freight locomotive on the Richmond, Fredericksburg and Potomac Railroad. Contours of the tires of the driving wheels were taken; those of the the Lehigh Valley Railroad were new and the others were in average working condition; those of the Richmond, Fredericksburg and Potomac Railroad were somewhat worn. Information on the counterbalance of the locomotives was not obtained, and the runs were made in a way that would give a distribution of tests throughout the revolution of the driver and thus the averages would mask the effect of counterbalance.

In Fig. 56 are given diagrams of the loaded cars—wheel spacing, capacities, and actual wheel loads. The loaded cars were weighed, and then each half was weighed separately; the wheel load was taken as one-fourth of the load on one truck. It is seen that the wheel loads of the different cars vary

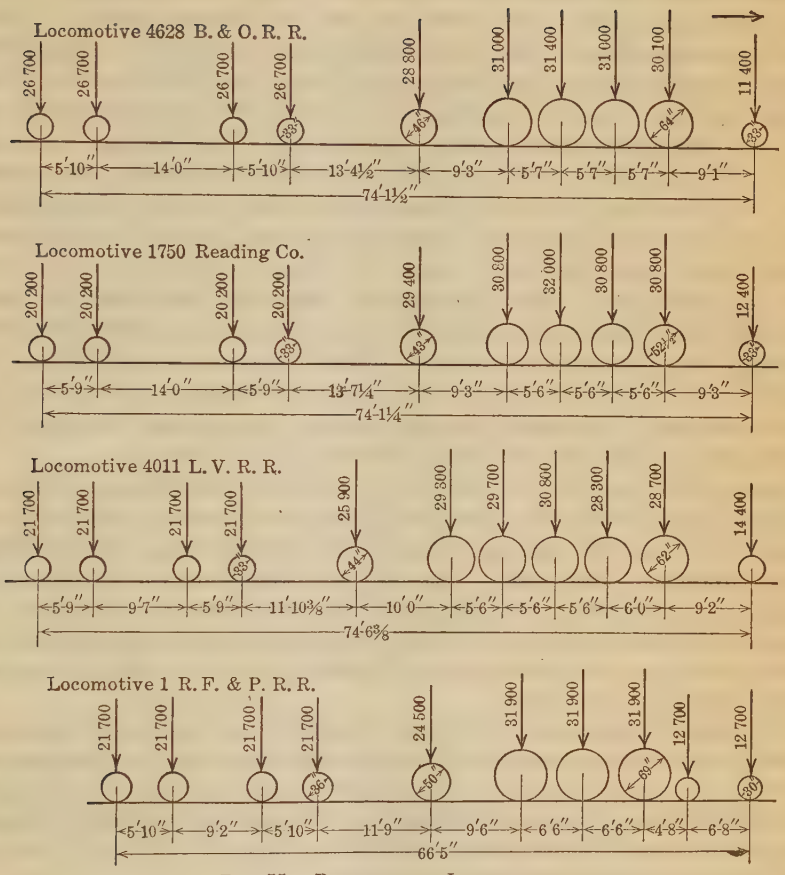


FIG. 55.—DIAGRAMS OF LOCOMOTIVES.

from 17 500 to 26 500 lb. The cars had been in service for some years; they had the variations that are to be found in cars in service. The wheels were not badly worn. The diameter of the wheels was 33 in.

It should be noted that the standard coning of the wheels of locomotives and cars of the Baltimore and Ohio Railroad is 1 in 13; that of all the other railroads is 1 in 20.

21.—Conduct of Tests and Reduction of Data.—The method of using the stremmatograph was similar to that described in the former progress reports. Four stremmatographs were placed on each rail, spaced about 5 ft. 6 in. apart, generally three tie spaces, the instruments on one rail being directly opposite instruments on the other rail. The 136-lb. rail on the Lehigh Valley Railroad required instruments of larger framed proportions, and only six instruments were available for this test. The driving mechanism used rotated the disks of all the instruments simultaneously. The correlation of a point on the record of one of the disks with the corresponding point of any other disk at the same moment was possible. It will be recalled that the stremmatograph records the strain at each edge of the base of rail on separate disks. The speeds used in these tests were usually 5 and 40 miles per hour; in the tests on the 10° curve of the Lehigh Valley Railroad, 5 and 35 miles per hour were used. Usually ten runs at each of the two speeds were made, the rotation being 5, 5, and 40 miles per hour on one set of disks and 5, 40, and 40 miles per hour on the next set. The speed was observed by a speedometer placed in the cab of the locomotive.

Impressions made by the wheels on copper wire laid across the rail were taken at most of the test locations. The wire generally was 0.1 in. in diameter, but smaller wire was used at the test locations on the Lehigh Valley Railroad. The head of the rail was punch-marked in advance to permit identification of the position of the wire with respect to the rail section. The impressions on the two rails were taken on separate runs, so the two sets may not be co-ordinate. Usually, tests were made on each rail for two runs at each speed.

The lateral movement of the head of the rail was measured by a special device. One end of a light steel rod was fastened to the rail in such way that a point at its end was held at a definite spot in the outside of the head about ½ in. below its top. The lateral movement of the rail was recorded by a needle on a smoked-glass disk placed on a rigid stake set about 5 ft. from the rail, and this disk was rotated as the train passed by.

The tilting of the rail with reference to its initial position was measured by another device. A bar placed under the two rails was fastened so that the rail could pivot about one of its edges. A needle-bar connected to the head of the rail at a definite place about ½ in. below the top of the rail, held a needle which made a record on a smoked-glass disk, the disk being fastened to a support at the end of the bar and rotated by an outside mechanism. The apparatus permitted vertical and lateral movement of the rail and measured only the movement due to tilting of the rail.

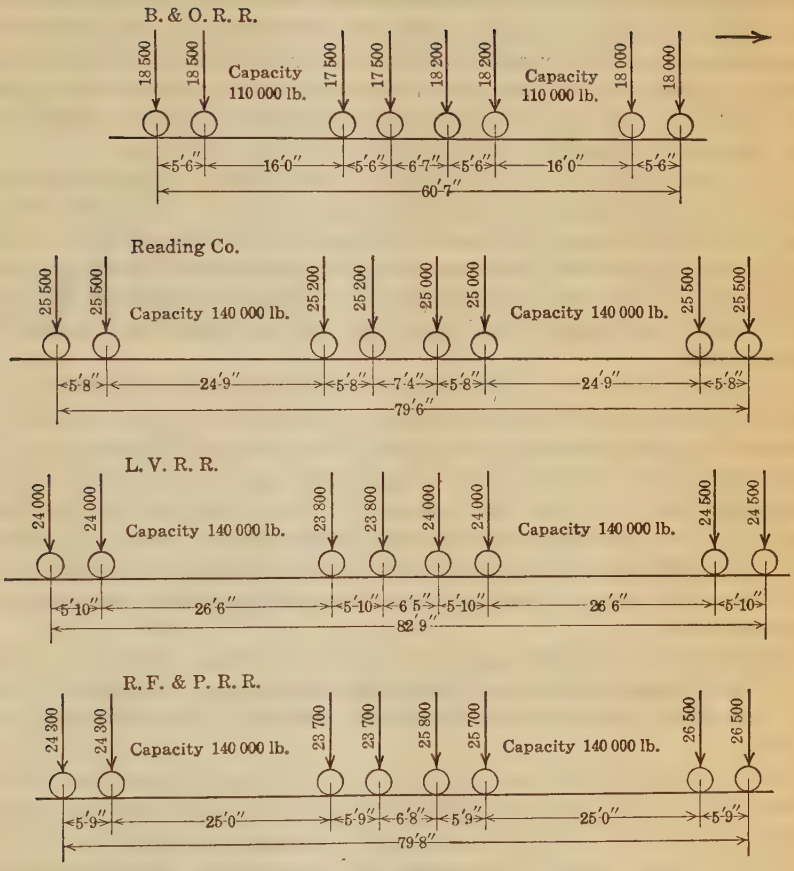


FIG. 56.—DIAGRAMS OF CARS.

The methods used on curved track were the same as on straight track. Care was taken to choose a location that was representative of the curve with respect to alignment, superelevation, cant, and gauge.

A method for determining the track depressions under load was developed. One end of a wooden bar was fastened to the base of the rail with a hinged connection. The other end was pivoted at the top of a stake 10 ft. from the rail. At a point two-thirds of the distance out from the rail an Ames dial gauge was placed between the bar and another stake. The Ames dial was read at and between the wheels as the locomotive and cars slowly passed by. Although this method gives the depression at only one point, both the test and the reduction of data are relatively rapid, and by making observations at three or more points it is believed that the results will be as satisfactory as those obtained with the longer and more tedious methods that have been used. Lack of time and opportunity prevented its use in more than a tentative way.

The process followed in the reduction of the stremmatograph data was the same as that described in the previous progress reports. About 135 000 readings with the microscope were made. Averages were calculated for the stresses at the inner edge and the outer edge of the base of rail and for the mean stress in the base of rail. The ratio of the stress at the outer edge of the base of rail to the mean stress in the base of rail was calculated for the individual observations.

As the stresses in the heavy rail were relatively small, it will be seen that an error of observation either instrumental or personal of a given amount will furnish a larger proportional error than will be found in lighter rail with the same wheel loads. Besides, as was found in the tests with heavy rail described in the first progress report, the chatterings on the record are greater with the heavy rail, due probably to the relatively greater mass of the rail as compared with the instrument and the shorter period of vibration. It is believed, however, that the errors of observation in these tests are about the same as in previous tests.

The records of the disks used in measuring the tilting and lateral movement of the rail were read by means of a low-power microscope. Readings were made for each wheel. The values are sufficiently accurate to give serviceable information.

22.—Stresses in Rail on Straight Track.—In Figs. 57 to 63 are given the mean stress in base of rail under and between wheels of the locomotive and the loaded freight cars. The values given are averages for both rails for all the runs at one speed, usually about ten runs being made. These stresses agree fairly closely with values calculated from the wheel loads and properties of the track by the method given in the first progress report. As compared with the stresses observed in rail of 85 and 90-lb. section, the mean stresses in base of rail are small, at 5 miles per hour averaging about 8 000 lb. per sq. in. under the drivers for the heavy rail and about 13 000 or 14 000 lb. per sq. in. for the light rail. At the higher speeds, the mean stress is also small. Under the wheels of the loaded freight cars, the mean stress in the heavy rail ranged from 6 000 to 10 000 lb. per sq. in. for a wheel load of 25 000 lb., at a speed of 5 miles per hour, nearly as great as that found under the drivers of the locomo-

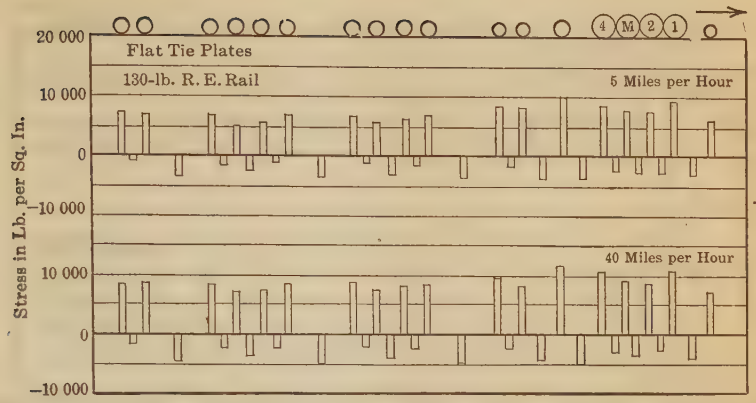


FIG. 57.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

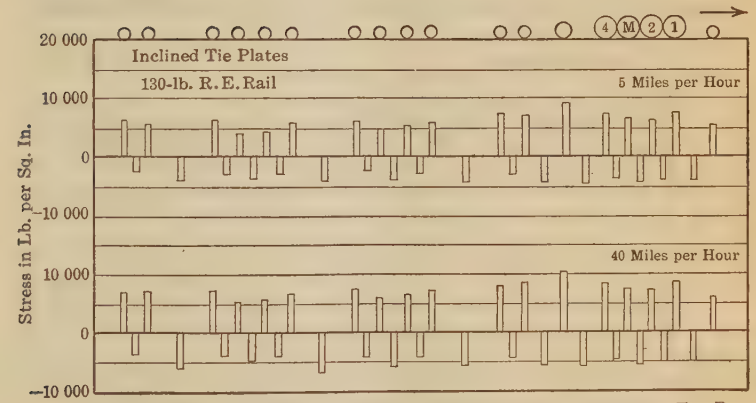


FIG. 58.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

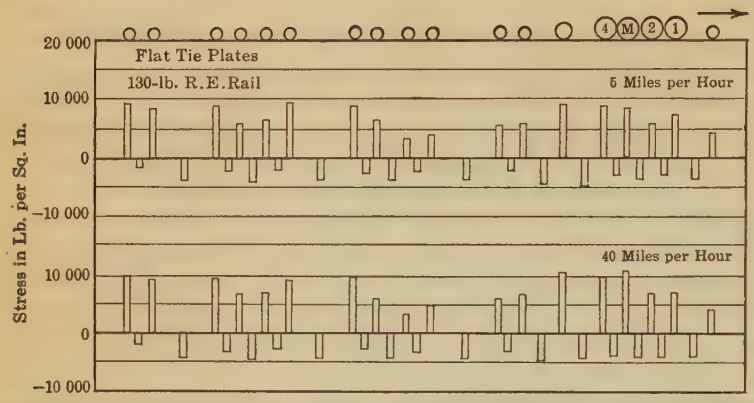


FIG. 59.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

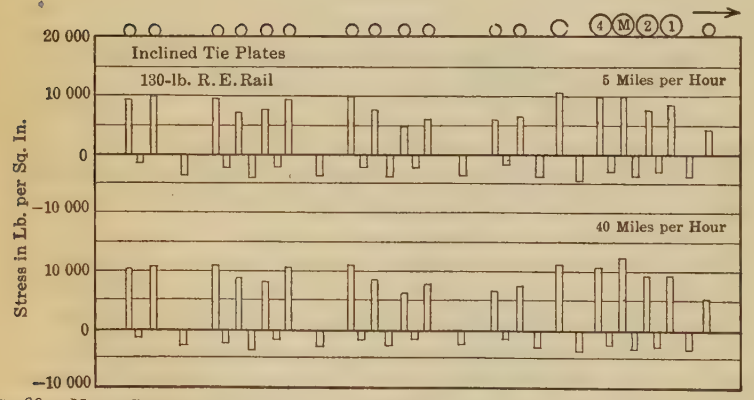


FIG. 60.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

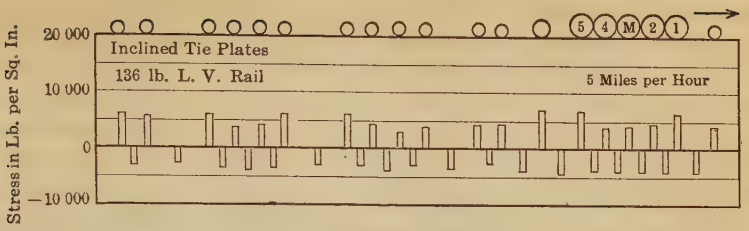


FIG. 61.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. SANTA FÉ TYPE LOCOMOTIVE AND CARS, LEHIGH VALLEY RAILROAD.

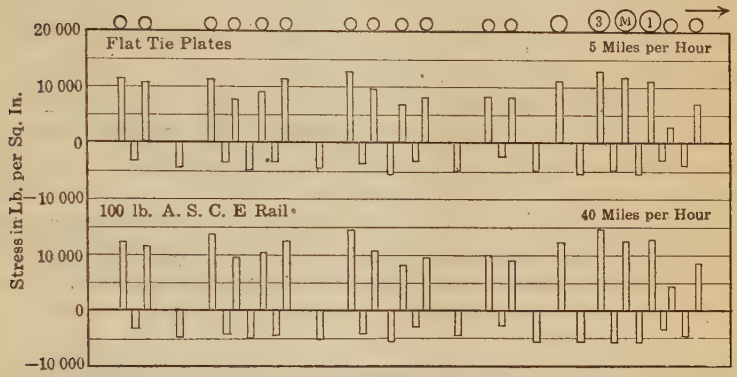


FIG. 62.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

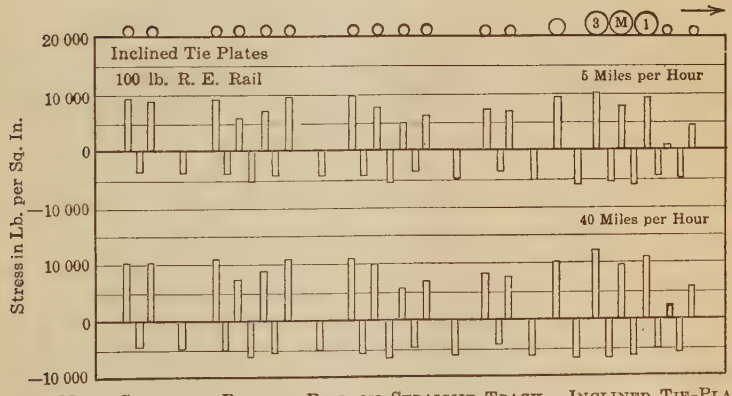


FIG. 63.—MEAN STRESS IN BASE OF RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

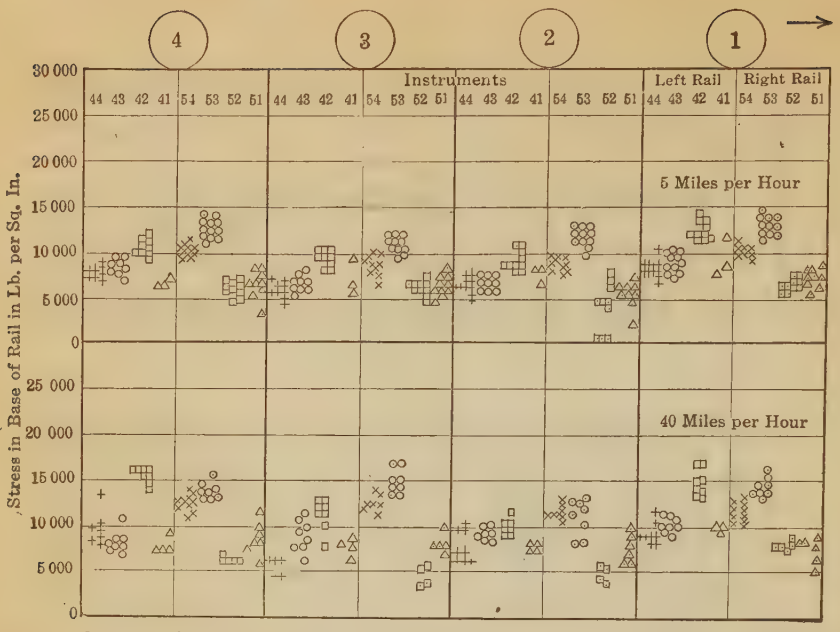


FIG. 64.—OBSERVED VALUES OF MEAN STRESS IN BASE OF RAIL AT THE EIGHT INSTRUMENTS ON STRAIGHT TRACK UNDER THE DRIVERS OF THE MIKADO TYPE LOCOMOTIVE OF THE BALTIMORE AND OHIO RAILROAD. FLAT TIE-PLATES.

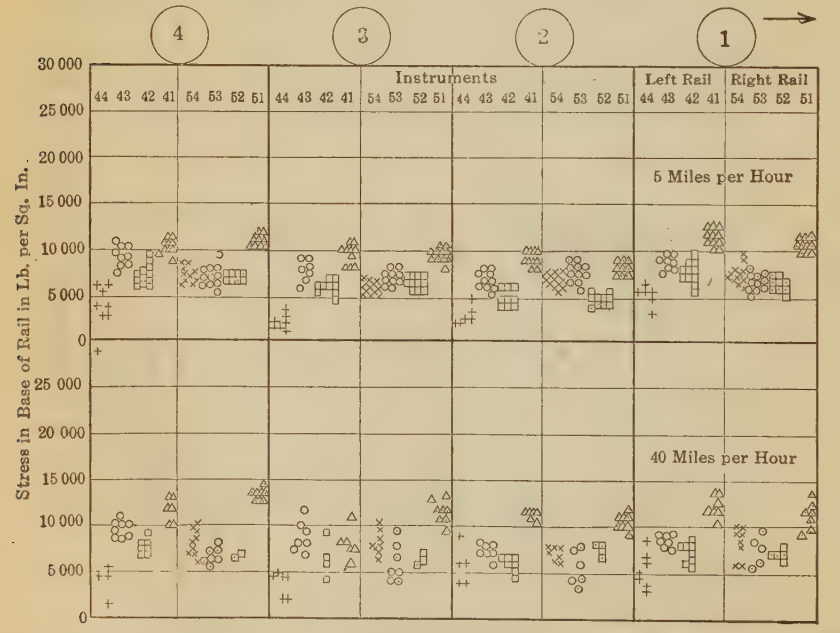


FIG. 65.—OBSERVED VALUES OF MEAN STRESS IN BASE OF RAIL AT THE EIGHT INSTRUMENTS ON STRAIGHT TRACK UNDER THE DRIVERS OF THE MIKADO TYPE LOCOMOTIVE OF THE BALTIMORE AND OHIO RAILROAD. INCLINED TIE-PLATES.

tive. It is seen from the figures that the stress under a wheel at the end of a car adjoining a tender or another car is smaller than that under the other wheel of the same truck, which has a longer spacing to the next wheel. This fact was brought out in Fig. 10* of the first progress report.

In Figs. 64 and 65 are given values of the individual observations of mean stress in base of rail for the drivers of the Mikado type locomotive on the Baltimore and Ohio Railroad as a sample of the variations in stress which are found with the heavy rail. Instruments 41, 42, 43, and 44 are on the left rail, and Instruments 51, 52, 53, and 54 are on the right rail, Instrument 41 being opposite Instrument 51, etc. The main belt of values seems to extend about 3 500 lb. per sq. in. on each side of the average line. The variation of values on the whole is similar to that found in tests previously made. It is evident that the condition of the track varies from instrument to instrument, the range of values at a given instrument being relatively small, in some cases not more than 1 000 lb. per sq. in. above and below the average for the instruments.

The relation between the stresses under the drivers and the position of the counterweight has not been studied, although the position with respect to the instruments was observed for every run. It is known that the position of counterweight was well distributed throughout the revolution of the drivers in the various runs so that the average values of stress given do not involve any effect of counterbalance.

Throughout the tests on straight track the average value of the mean stress in base of rail for all the wheels of locomotive and cars for one rail did not differ much from that for the other rail. Some differences were found, however, in the two rails of the Richmond, Fredericksburg and Potomac Railroad, due in part in one case to one rail being lower than the other.

The average increase in mean stress in base of rail on straight track for a change from 5 to 40 miles per hour for the locomotives ranged from 14 to 17% on the three railroads and for the cars from 10 to 25%, values for individual wheels in some cases being much higher. In some cases the increase in stress differed in the two rails. In some locations the left rail gave the higher increase; in others, the right one. The difference between the higher value and the average of the two rails ranged from 1 to 10 per cent. It is seen that an increase of 10% in the stress means a change generally less than 1 000 lb. per sq. in. Such variations may easily be due to differences in track conditions.

23.—Lateral Bending Stresses in Straight Track and Ratio of Stress at Outside Edge to Mean Stress in Base of Rail.—In all tests of track the rail has been found to bend laterally, both inwardly and outwardly of the track. If the lateral curvature is outward at a wheel it is usually inward at a point between wheels, and *vice versa*. This flexural action is similar to the vertical bending of the rail, convex downward at a wheel and upward between wheels. The lateral bending of the rail in the sense here used should be distinguished from the larger outward or inward movement of the rail as a whole or for distances greater than, say, 40 in. for a group of wheels, in just the same way that the bending of a rail in the vertical plane must be distinguished from the

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1214.

depression of the rail and track for the length of a group of wheels. The bending of a rail as a beam, both vertical and lateral, is of a smaller order of magnitude than the vertical and lateral movement of the rail. The lateral bending stresses may have one source and the lateral movement another.

Figs. 66 and 67 are given as examples of the values of the stresses at the two edges of the base of rail, the one on the Reading Company showing relatively small lateral bending and the other on the Richmond, Fredericksburg and Potomac Railroad, relatively large. The values include every readable record in all the runs and all the instruments. On the track of the Reading Company, Instruments 41, 42, 43, and 44 are on the left rail and Instruments 51, 52, 53, and 54 are on the right rail, Instrument 41 being opposite Instrument 51, etc., while on the Richmond, Fredericksburg and Potomac Railroad, these two groups of instruments were placed on the right rail and the left rail, respectively. It is seen that the lateral bending stresses may be large or small. The heavier sections may be expected to have lower lateral bending stresses than the lighter ones, although proportionally they may not differ greatly.

The difference between the stress in the outer edge of the base of rail and the mean stress (average of the stresses in the two edges of the base of rail) may be taken as the stress due to lateral bending of the rail and to be indicative of the amount of lateral bending. If it is positive, the rail is bent outward; if negative, inward. It is found that there is a great variation between individual values of these lateral bending stresses and the average value, and there is need of consideration of the amount and source of such variation.

If the stress at the outside edge of the base of rail is divided by the mean stress, the resulting ratio may be used as an indication of the condition and amount of lateral bending. If this stress ratio is greater than unity, the bending is outward; if less, inward. The range of the values of this ratio will indicate the variations in lateral bending. It should be borne in mind, however, that for the heavier rail a given ratio will mean a relatively smaller lateral bending stress than that in a lighter rail for the same stress ratio.

In Figs. 68 to 75, values of the stress ratios have been plotted for both rails, for each instrument, and for each wheel of the locomotives and cars. Space permits giving diagrams for only a small part of the work, but Figs. 68 to 75 will give some knowledge of the ranges in values and the variations that may be expected at different points on the track and for different runs of the train.

It will be seen that the stress ratio found at a given instrument varies from run to run. Examples of such variations in the range are from 0.40 to 0.70, 0.65 to 0.90, 0.85 to 1.15, 1.00 to 1.45, 1.20 to 1.30, and 0.85 to 1.20. The range in general is 0.15 or so on each side of the average value at the given instrument. Sometimes the range is less than this; occasionally it is more.

The foregoing ranges are for the same wheels of one locomotive or one car. Different wheels give different values of the ratio. Different wheels may give values (averages) at the same instrument varying by as much as 0.20, but usually not more than 0.10. It was found in the tests on the Chicago, Mil-

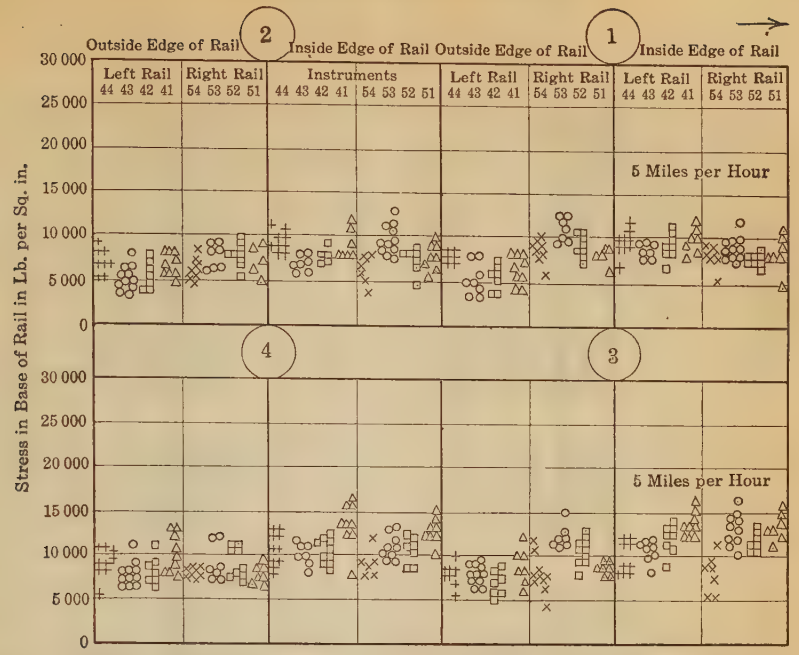


FIG. 66.—OBSERVED VALUES OF STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF RAIL ON STRAIGHT TRACK UNDER THE DRIVERS OF THE MIKADO TYPE LOCOMOTIVE OF THE READING COMPANY. INCLINED TIE-PLATES.

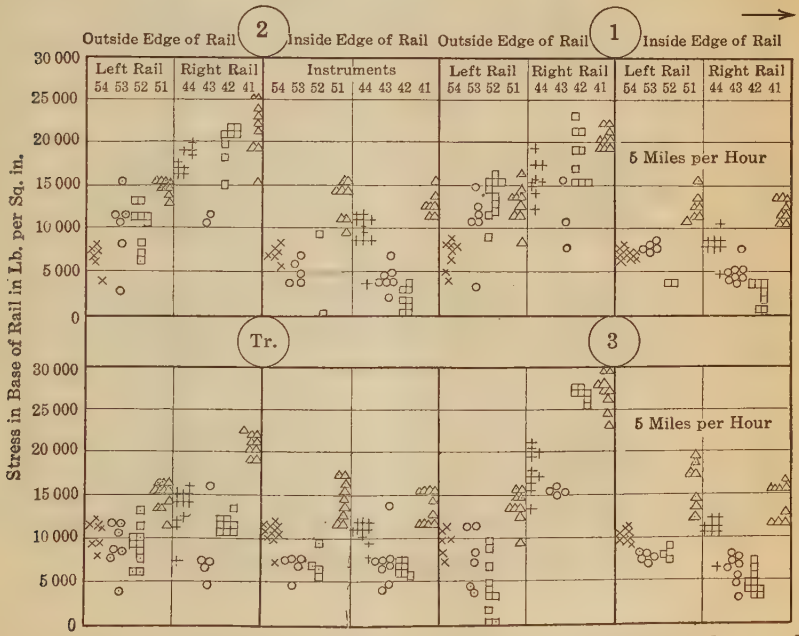


FIG. 67.—OBSERVED VALUES OF STRESS AT INSIDE EDGE AND OUTSIDE EDGE OF BASE OF RAIL ON STRAIGHT TRACK UNDER THE DRIVERS AND TRAILER OF THE PACIFIC TYPE LOCOMOTIVE OF THE RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD. FLAT TIE-PLATES.

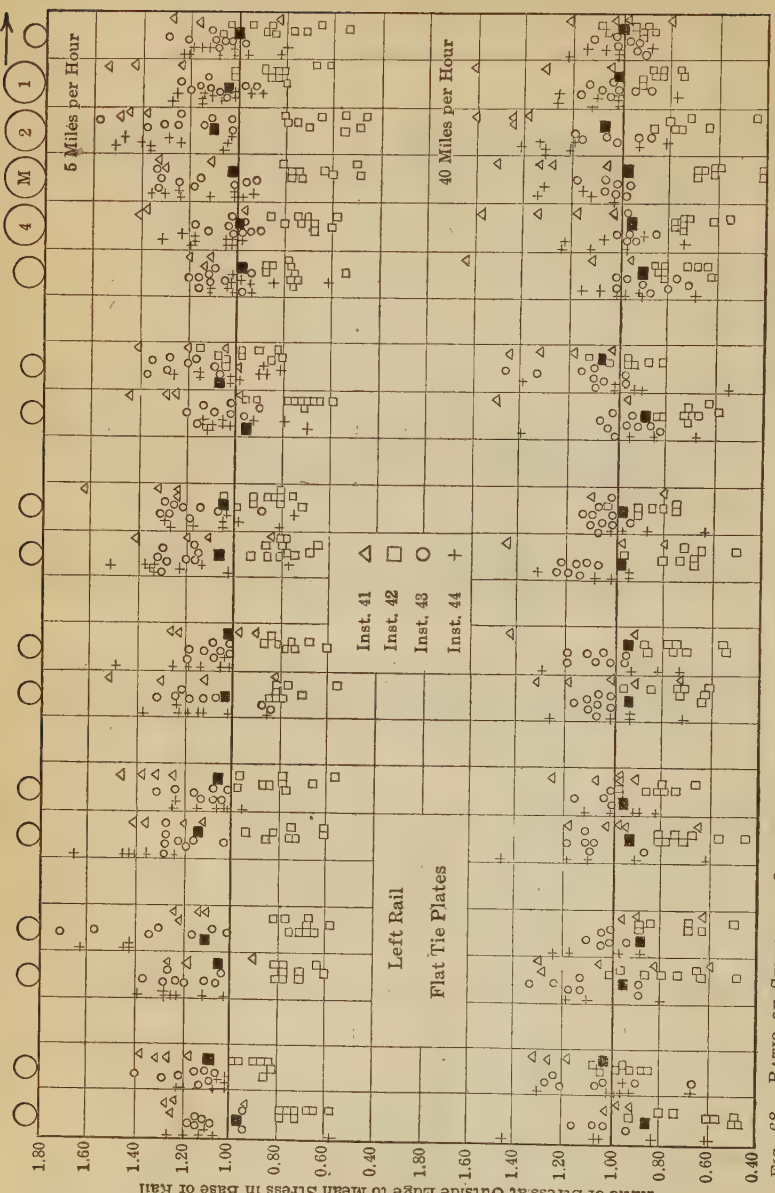


FIG. 68.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF LEFT RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

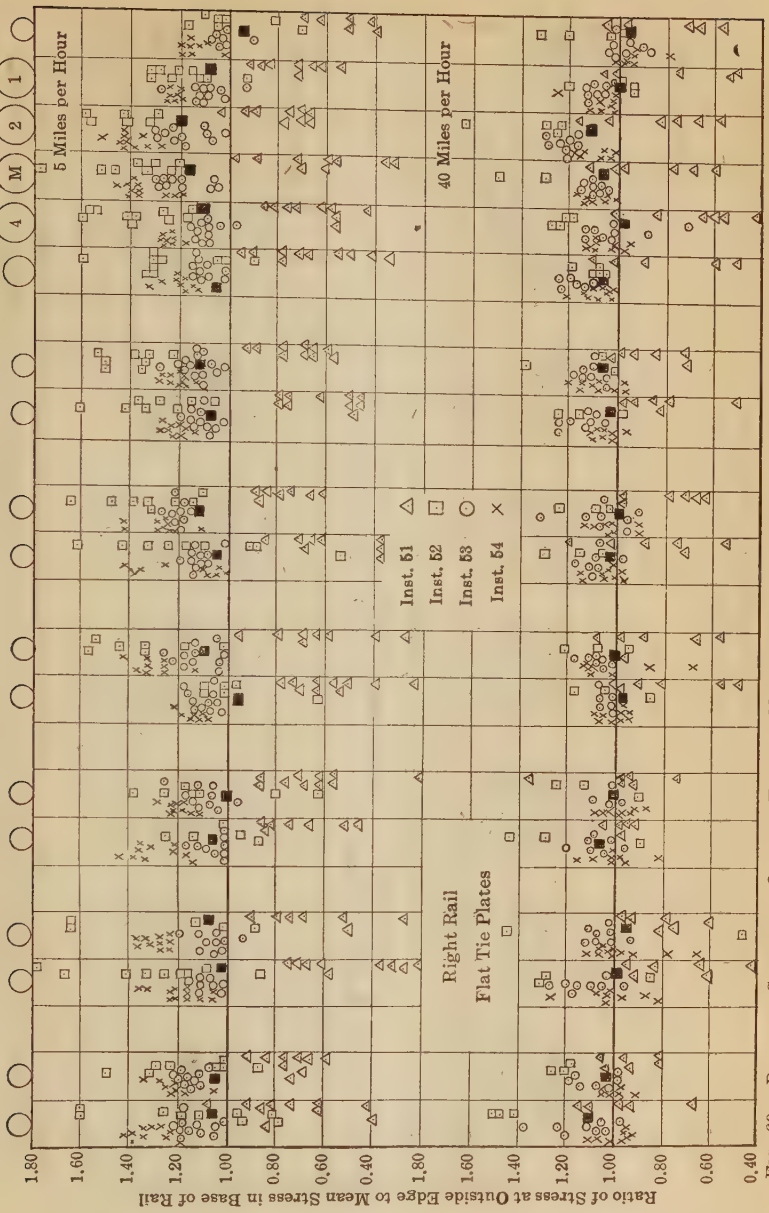


FIG. 69.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RIGHT RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

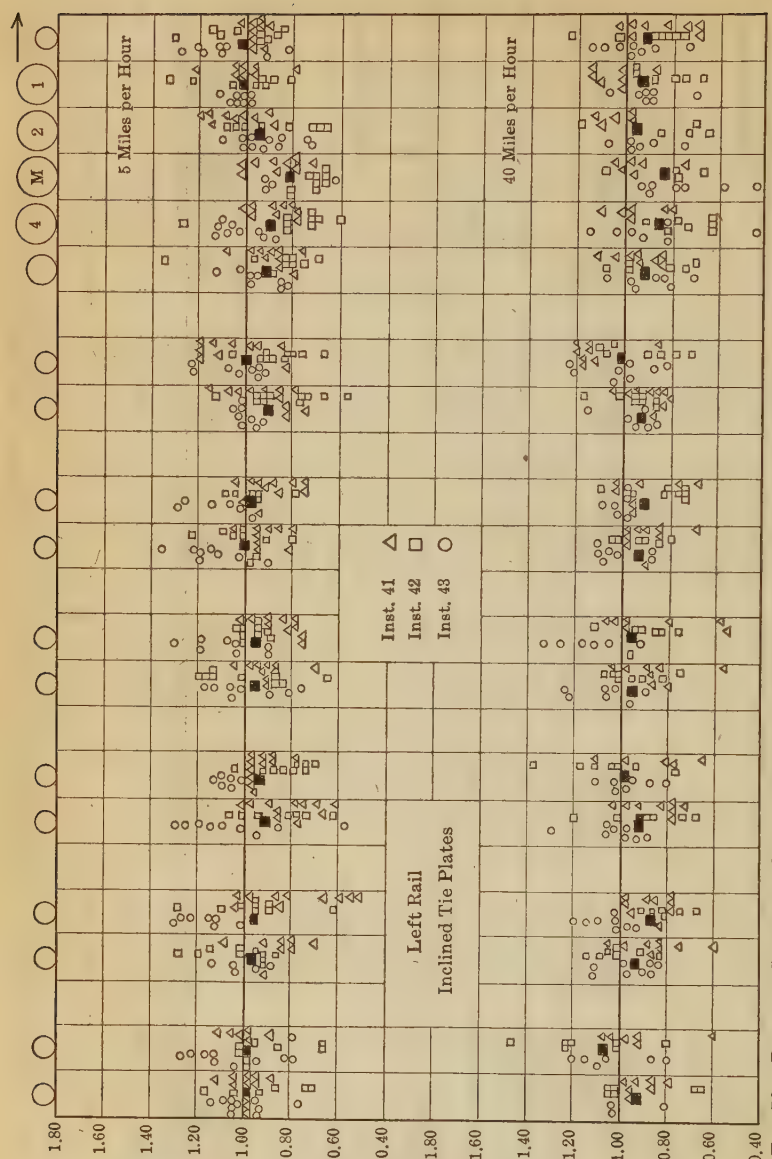


FIG. 70.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF LEFT RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

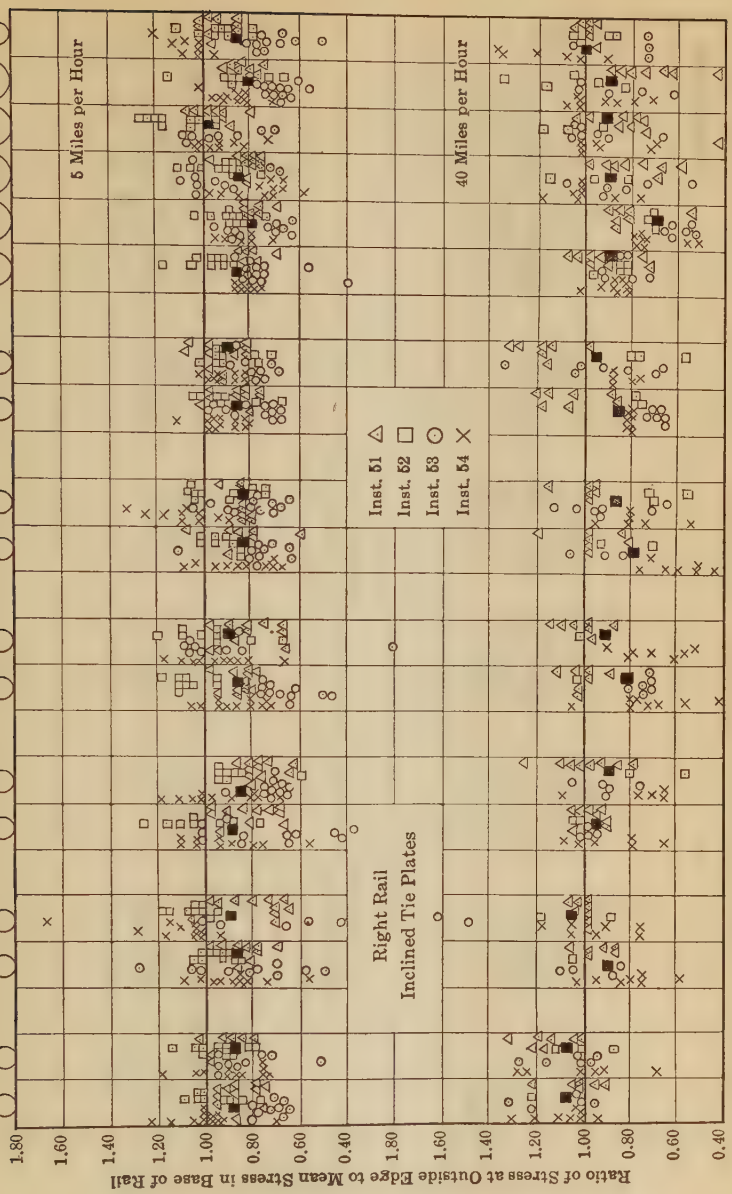


FIG. 71.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RIGHT RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

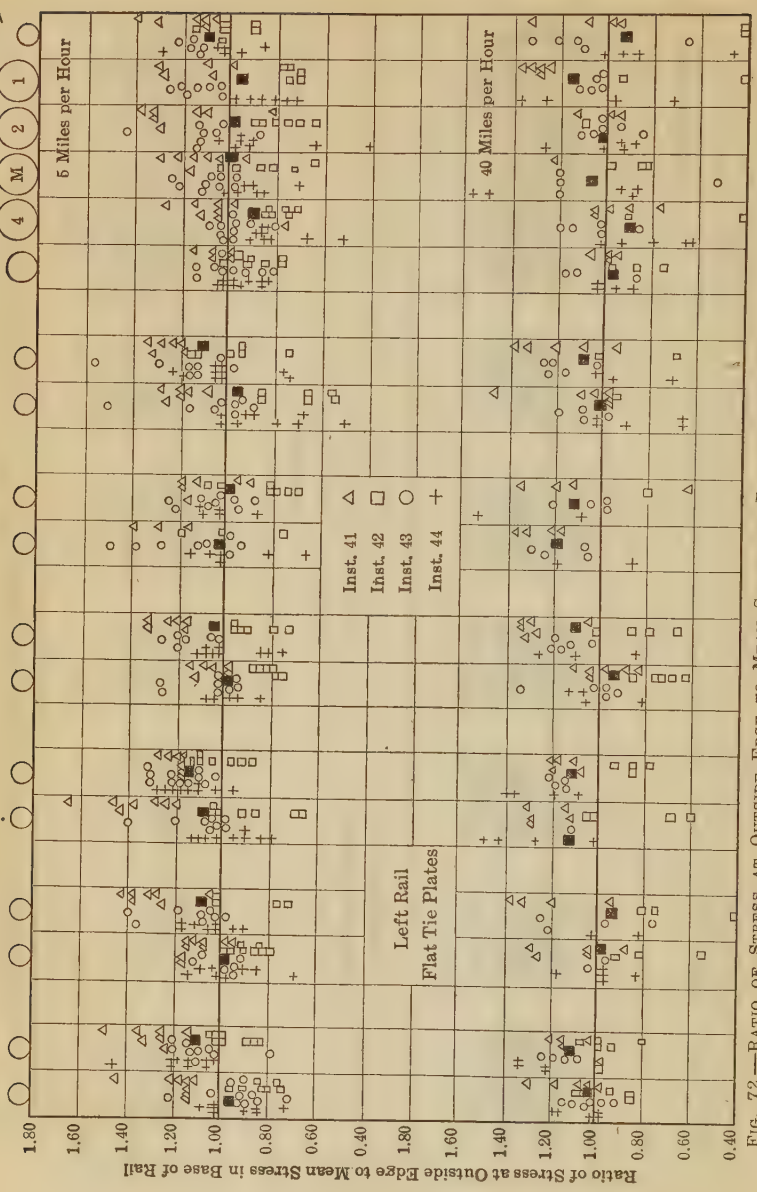


FIG. 72.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF LEFT RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

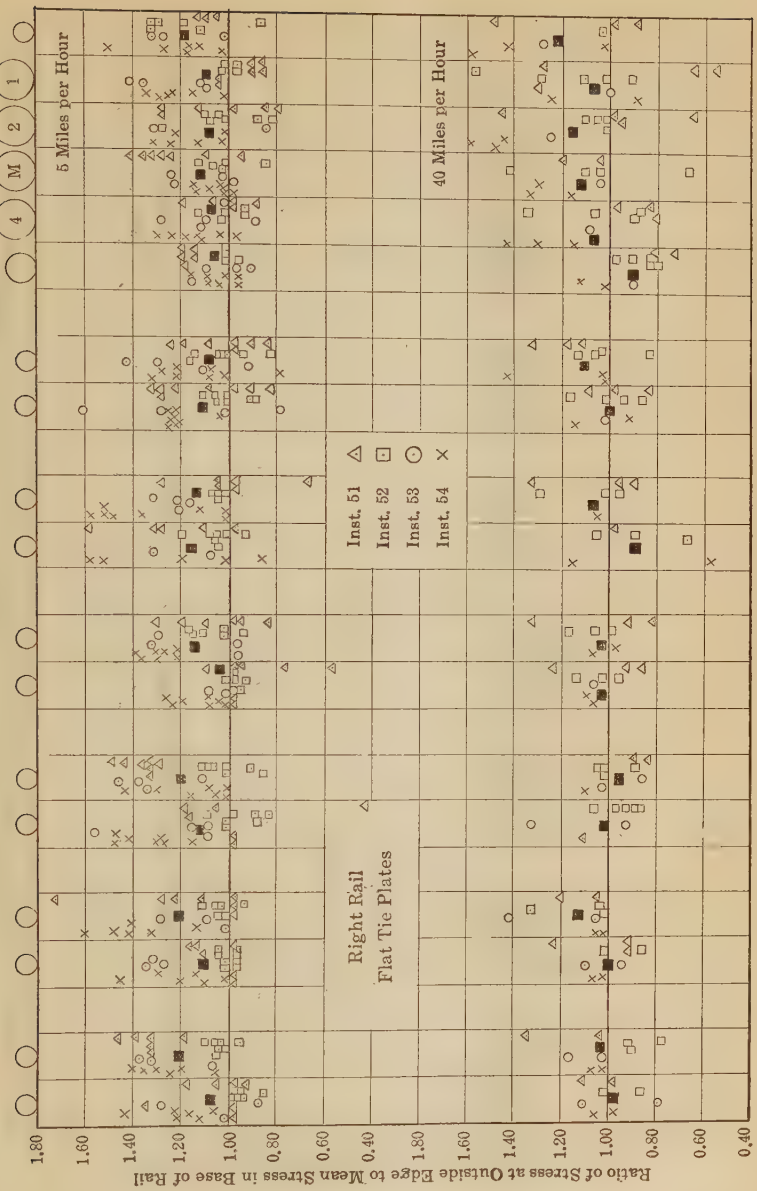


FIG. 73.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RIGHT RAIL ON STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

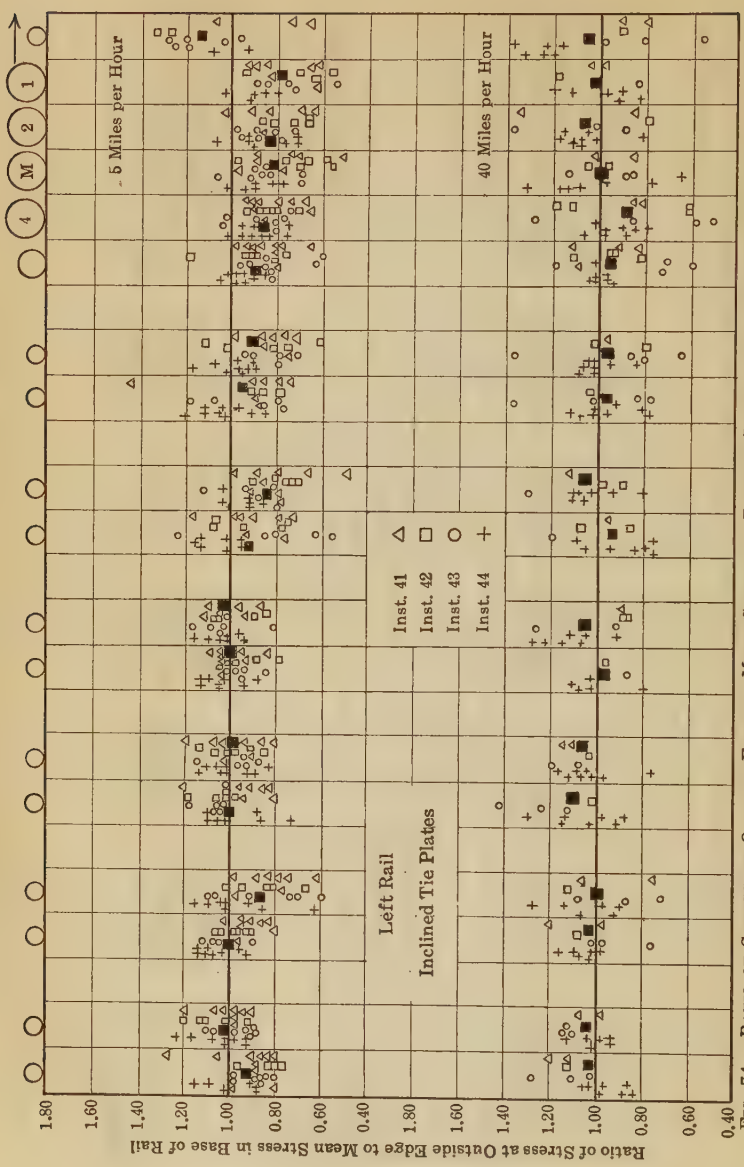


FIG. 74.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF LEFT RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

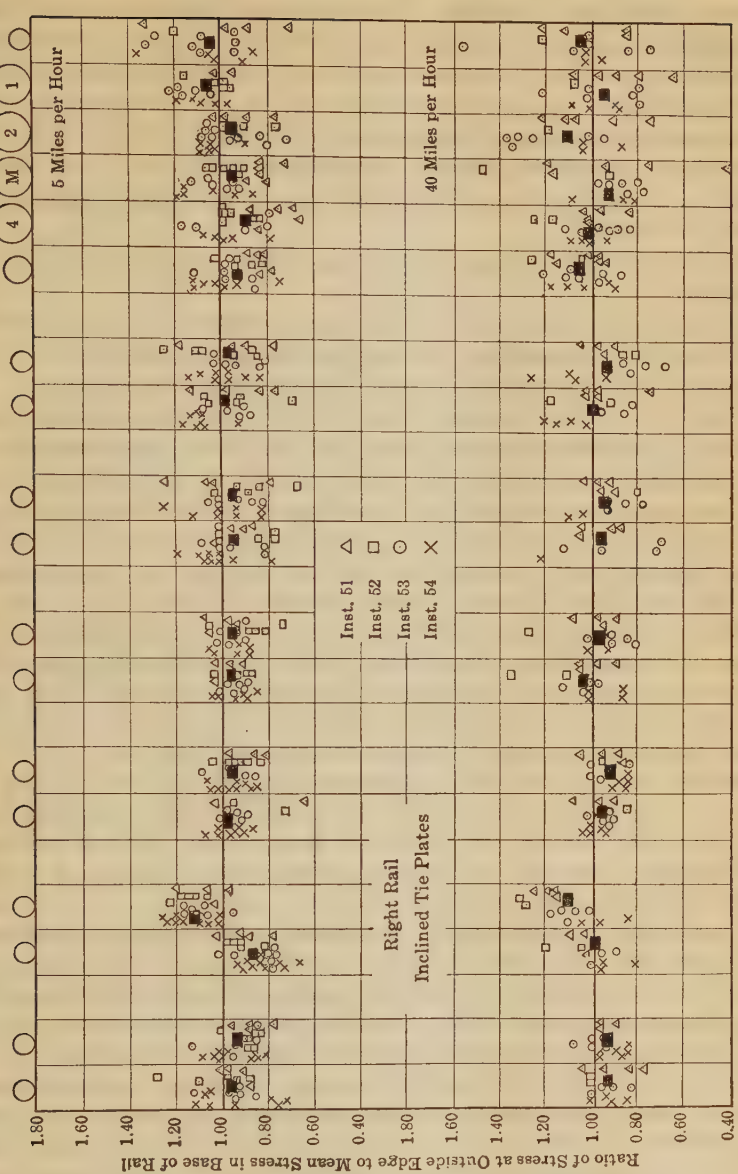


FIG. 75.—RATIO OF STRESS AT OUTSIDE EDGE TO MEAN STRESS IN BASE OF RIGHT RAIL ON STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, READING COMPANY.

waukee and St. Paul Railway that wheels of one locomotive gave quite different ratios at a given instrument than those of another locomotive.

It was found that the average stress ratio at one instrument may differ considerably from that at another. While part of this may be due to instrumental differences, it is evident that the greater part is due to differences in track conditions, and that variations in the rail support must be expected to produce variations in lateral bending of the rail from point to point along the rail. Averages for the several instruments on the two locations on the Baltimore and Ohio Railroad and the two of the Reading Company show that the stress ratio may vary more than 20% from the average of all the instruments at a given location.

The values for points along one rail may differ considerably from those for the other.

The foregoing discussion relates to the range in values that may be expected on good track with locomotives and cars in good condition. It will be best next to consider the average values found under different conditions.

In Table 15 are given general values of the average ratio of the stress at the outside to the mean stress in base of rail for the two rails separately for all the tests for the two speeds used. The values are not always averages, and values both larger and smaller than those given were found under individual wheels. The data given by Instrument 44 on the track of the Baltimore and Ohio Railroad, laid with inclined tie-plates, were not included in the averages for the left rail, since there was internal evidence that the records were not fully reliable. A study of the data of all the tests showed that there was a belt of values of the individual ratios for any wheel that extended about 0.20 on each side of the average value of the ratio, generally with only a few values outside this belt; that is, if the average of all the values under a given wheel was 0.90, the range of most of the observations would be from 0.70 to 1.10; if the average was 1.20, the main range was from 1.00 to 1.40. In some test locations for one rail or both, however, the averages at one instrument varied considerably from those at the others, the average ratio for the instrument being 0.20 away from that of the other instruments, indicating a different condition of track at these points.

Although there is a diversity in the ratios for the different locations for one rail from the other, and for different instruments, it will be seen from Table 15 and the diagrams that with flat tie-plates the ratio of the stress at the outside edge to mean stress in base of rail is generally greater than unity and is greater than that found with the inclined tie-plates. The ratio with the inclined tie-plate is near unity, or below unity. In general, then, the lateral bending of the rail is outward when laid with flat tie-plates, and inclined tie-plates generally give little average bending or a little inward bending. These values of the stress ratios, however, are averages, and marked variation on each side of the average was found. The condition of the track itself is an element in lateral bending; in the case of the right rail of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates, but actually being canted outwardly 1 in 30 at the test location, the average stress ratio

at all the wheels was in 1.25, the greatest average outward bending found under any of the test trains.

It may be noted that although the average values at the higher speed are somewhat closer to unity than those at 5 miles per hour, the range in ratios is much the same and the variation in results for different runs, different wheels, and different instruments is as great.

TABLE 15.—Ratio of the Stress at the Outside Edge to Mean Stress in Base of Rail on Straight Track.

(A value greater than unity means outward bending; less than unity, inward bending.)

Location.	Speed, in miles per hour.	Left Rail.			Right Rail.		
		Cars.	Tender.	Locomotive.	Cars.	Tender.	Locomotive.
Baltimore and Ohio Railroad:							
Flat tie-plates	5	1.06*	1.03*	1.02*	1.05*	1.07*	1.11*
	40	0.94*	0.98*	0.99*	1.02*	1.03*	1.03*
Inclined tie-plates	5	0.96†	0.97†	0.94†	0.88	0.88	0.87
	40	0.95†	0.95†	0.90†	0.96	0.86	0.88
Reading Co.:							
Flat tie-plates	5	1.05	1.01	0.98	1.14	1.13	1.11
	40	1.05	1.09	1.00	1.02	1.02	1.09
Inclined tie-plates	5	0.98	0.91	0.89	0.97	0.97	0.97
	40	1.04	0.99	1.00	0.99	0.96	1.02
Lehigh Valley Railroad:							
Inclined tie-plates	5	0.99	1.00	0.93	0.92	0.97	0.97
Richmond, Fredericksburg and Potomac Railroad:							
Flat tie-plates	5	1.10	1.14	1.01	1.29	1.27	1.32
	40	1.09	1.08	1.09	1.24	1.33	1.35
Inclined tie-plates	5	1.04	1.02	1.04	1.03	1.13	1.11
	40	0.97	0.93	1.00	1.07	1.08	1.18

* Includes one instrument much below average.

† Instrument 44 omitted.

24.—The Tilting and Lateral Movement of the Rail Under Load.—Straight Track.—It was found by means of the apparatus described under Article 21, "Conduct of Tests and Reduction of Data", that on straight track the rail itself tilted outwardly or inwardly as the wheels passed by and that the head of the rail moved outwardly or inwardly, and even that the base of the rail had a slight lateral movement in addition to that due to the lateral bending of the rail already discussed. Figs. 76 and 77 give the lateral tilting of the rail. The measurements were made at the side of the head ½ in. below the top of the rail and represent, in inches, the lateral tilting of the rail for the vertical distance of this point above the base, irrespective of the lateral movement of the base. Figs. 78 and 79 give the lateral movement of the same point on the head of the rail. For both tilting and lateral movement, outward movement is called positive and inward movement negative.

Outward movement tends to widen the gauge and inward movement to narrow it. The general movement of the base of rail may be found by sub-

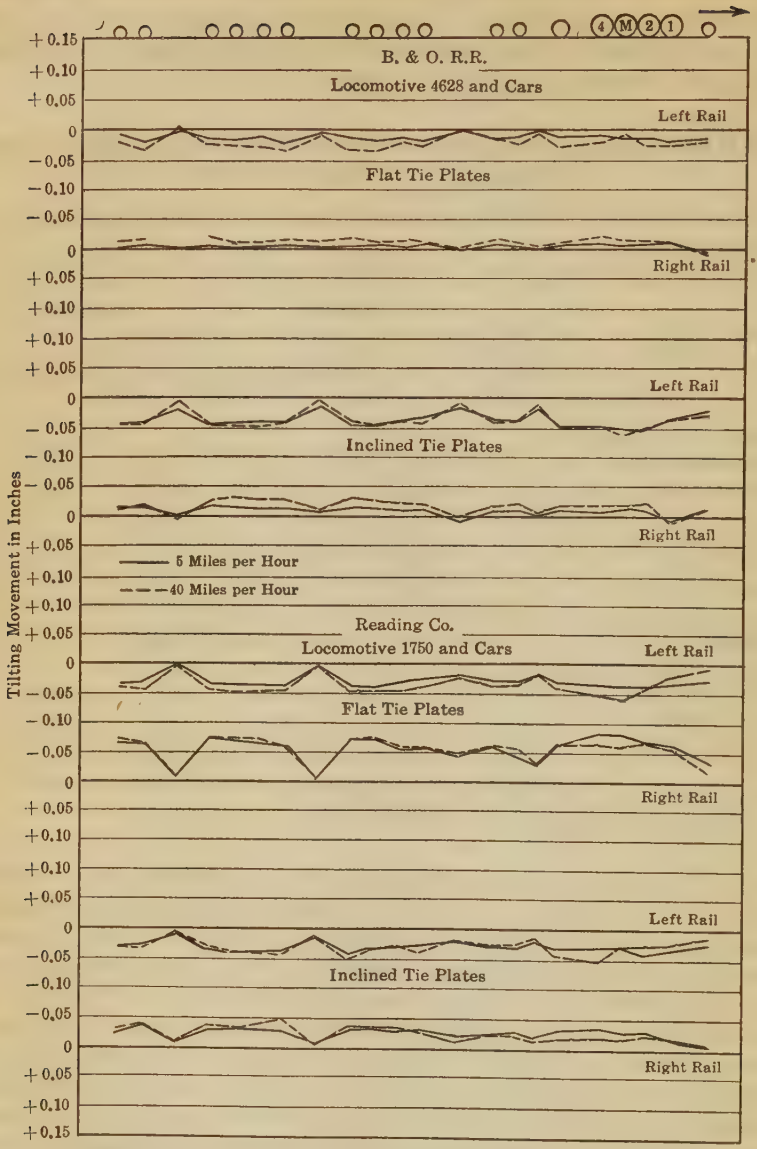


FIG. 76.—TILTING MOVEMENT OF RAIL ON STRAIGHT TRACK. MIKADO TYPE LOCOMOTIVES AND CARS, BALTIMORE AND OHIO RAILROAD AND READING COMPANY.

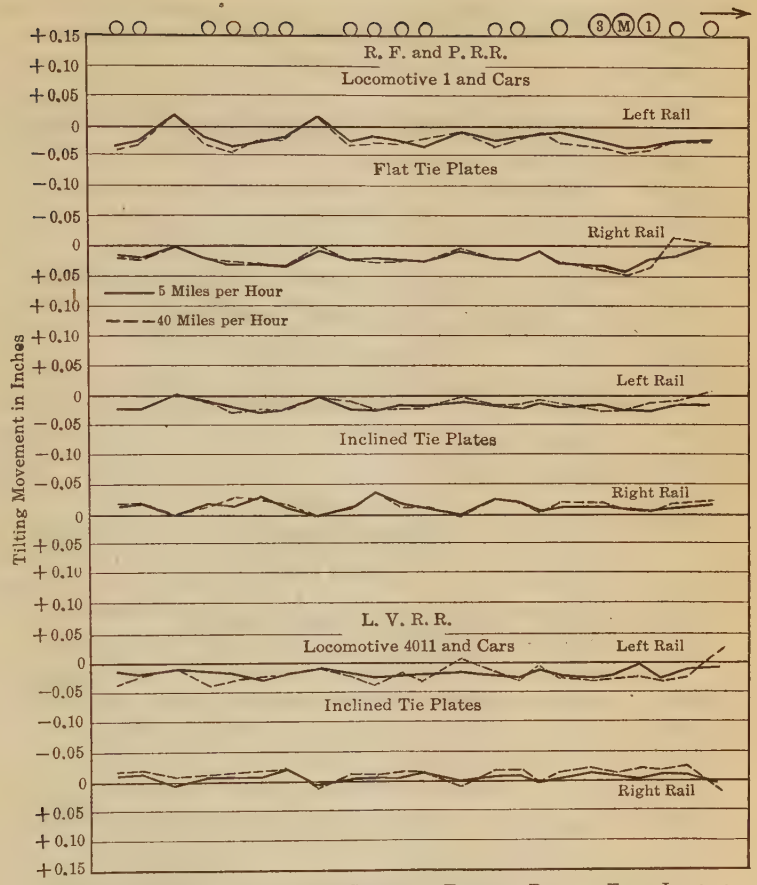


FIG. 77.—TILTING MOVEMENT OF RAIL ON STRAIGHT TRACK. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD. SANTA FÉ TYPE LOCOMOTIVE AND CARS, LEHIGH VALLEY RAILROAD.

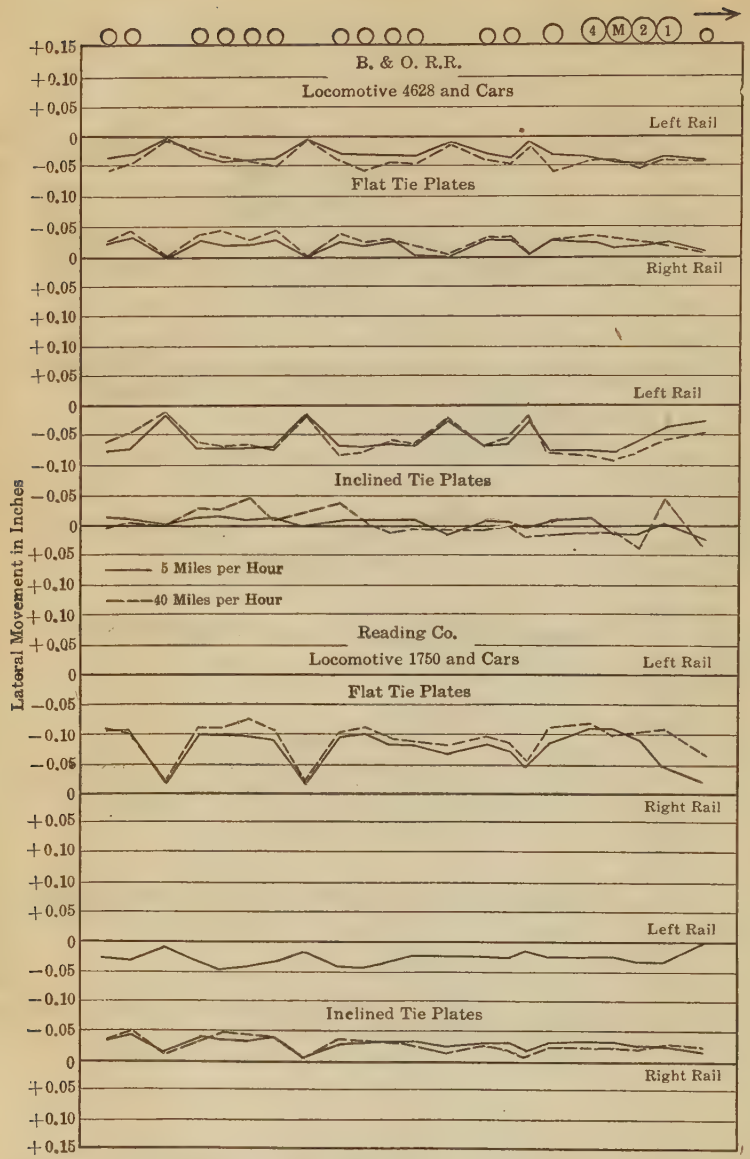


FIG. 78.—LATERAL MOVEMENT OF HEAD OF RAIL ON STRAIGHT TRACK. MIKADO TYPE LOCOMOTIVES AND CARS, BALTIMORE AND OHIO RAILROAD AND READING COMPANY.

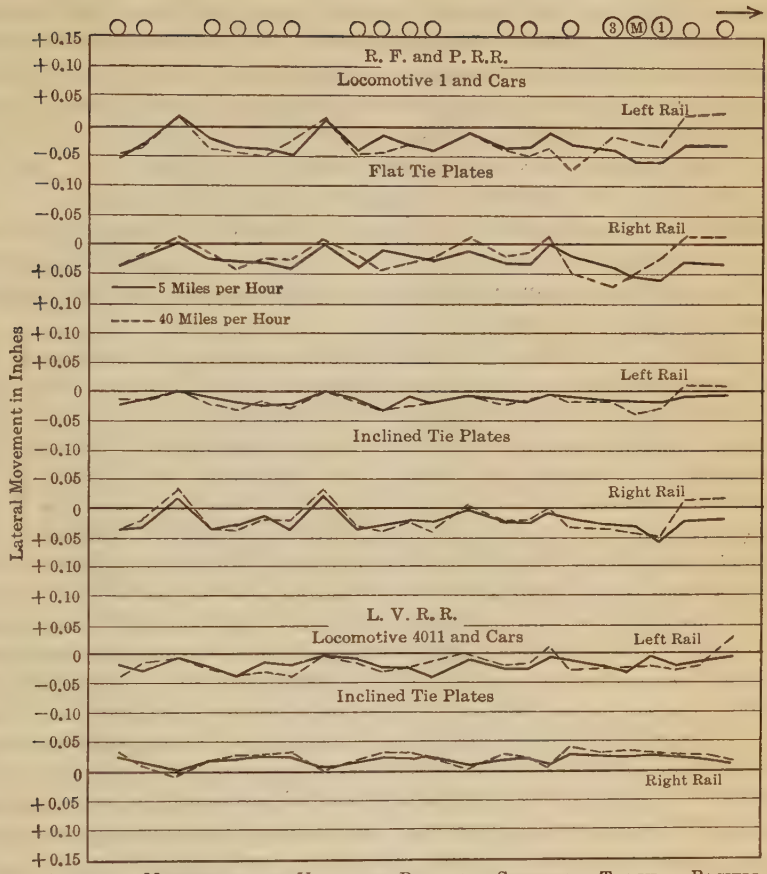


FIG. 79.—LATERAL MOVEMENT OF HEAD OF RAIL ON STRAIGHT TRACK. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD. SANTA FÉ TYPE LOCOMOTIVE AND CARS, LEHIGH VALLEY RAILROAD.

tracting algebraically the amount of the tilting movement from that of the movement of the head of the rail. For both movements, the values given are averages of two to four runs.

It will be seen that the lateral movement of the head is generally greater than that of the tilting action, the former ranging from, say, 0.04 to 0.10 in., and the latter from 0.02 to 0.06 in. The movement of the rail was inward in all cases except in that of the right rail of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates, which was 2 in. lower than the left rail, and the right rail of the same railroad laid with inclined tie-plates. The measurements made on the rails in the tests of straight track of the Chicago, Milwaukee and St. Paul Railway showed a general outward movement of the head. It should be noted that all the tie-plates used on the track tested on the Eastern railroads projected farther on the outside of the rail than on the inside, the eccentricity ranging from 0.15 to 0.50 in.

Although the lateral movement of the rail is small, its presence and direction are an indication of the tilting of the tie-plate due to greater pressure at one end than at the other, and hence points to conditions that may be important as affecting track maintenance.

25.—Position of Bearing of Wheel on Rail on Straight Track.—As has already been described, a copper wire laid across the rail was pulled a short distance between the passage of one wheel and the next and again at the next interval, the wire flattened by the wheel giving information on the position of the bearing of the wheel on the rail, the impression left on the wire by punch-marks made on the rail in advance permitting identification of the position of the wire on the rail. In Figs. 80 to 84 are plotted results of tests with the copper wire. The width of the flattened portion at various points along the length of the impression was measured and the initial diameter of the wire was subtracted, an arbitrary procedure; the remainder was plotted above the rail in the diagrams. The plot so made does not measure the intensities of pressure of wheel on rail, but the position of the greatest ordinate or perhaps the center of gravity of the area may be considered to be the point of greatest pressure, and the bearing contact may be expected to extend a short distance on either side of this point, but not to the limit of the area so plotted. The intensity of pressure will be greatest at the point of greatest ordinate and decrease rapidly each way, the width of the bearing being much less than represented in the diagram. It should also be noted that an impression shown at the gauge side of the head may mean merely that the flange was close to the rail; only a considerable width of the area is indicative of flange pressure.

For the test location of the Baltimore and Ohio Railroad, with inclined tie-plates (cant of rail 1 in 15 and 1 in 17), the center of pressure between wheel and rail for both locomotive and cars was at or near the middle of the the head of the rail. The brightness was spread well over the full width of the head of the rail. With the flat tie-plates (rails canted inwardly about 1 in 60), the center of bearing for both locomotive and cars was on an average about 0.6 in. away from the middle of the head and toward the gauge side.

As this rail had been on flat tie-plates only a few days, the position of the bright spot is not significant.

On the track of the Reading Company, with inclined tie-plates (cant of rail about 1 in 17 and 1 in 16), the center of bearing was slightly outside the middle of the head. On this stretch of track the rails were worn bright from the gauge side nearly to the outer edge of the top of the rail. With the flat tie-plates (rail almost vertical), the bearing is seen to be well inside the middle of the head of the rail. On this stretch the rails were worn bright about two-thirds of the way across the head from a point near the gauge side.

On the Lehigh Valley Railroad, with inclined tie-plates (cant of rail 1 in 18), the center of bearing although variable under the various wheels of the locomotive and cars, has an average position slightly inside the middle of the head. It should be noted that the rails on this track had been worn down considerably, as shown by Fig. 84.

On the Richmond, Fredericksburg and Potomac Railroad, with inclined tie-plates (cant of rail 1 in 18), the center of bearing on the right rail was somewhat inside the middle of the head. No impressions were taken on the left rail, but it showed bright wear over all the head. With the flat tie-plates (rail vertical), the center of bearing on the left rail was found to be somewhat inside the middle of the head, about 0.3 in. The right rail (canted 1 in 30 outwardly and 2 in. lower than the left rail) was bright from the gauge side to a point not as far as the middle of the head. The rails on this stretch were worn down somewhat more than those of the Baltimore and Ohio Railroad and the Reading Company.

In Column (3) of Table 16 is given more accurately the estimated average position of the center of the wheel bearings for the several test locations. The values were taken mainly from the averages of the impressions of copper wire when available and otherwise from the appearance of the rail and other observations. The values are given in inches, the positive direction being inward from the middle of the head of the rail.

26.—Discussion of Canting Rail and of Unsymmetrical Tie-Plates on Straight Track.—In discussing the tests a distinction should be made between the purpose and effect of canting the rail and of using unsymmetrical tie-plates. It will be evident that the two give separate and apparently not closely related effects.

A study of the results shows that for the rails laid on inclined tie-plates the averages of the ratios of stress in outer edge to mean stress in base of rail are either close to unity or less than unity, and that the averages of the ratios for the rails laid with flat tie-plates is greater than unity and greater than those found in rails laid with the inclined tie-plates. In other words, the lateral bending of the rail canted inwardly of the track on the average is very small or the bending is inward, the latter being found when the rail is canted more than 1 in 20; and for rail laid vertically the outward lateral bending stresses on the average are quite marked. It may be expected that the maintenance of track affected by lateral bending of rail will be

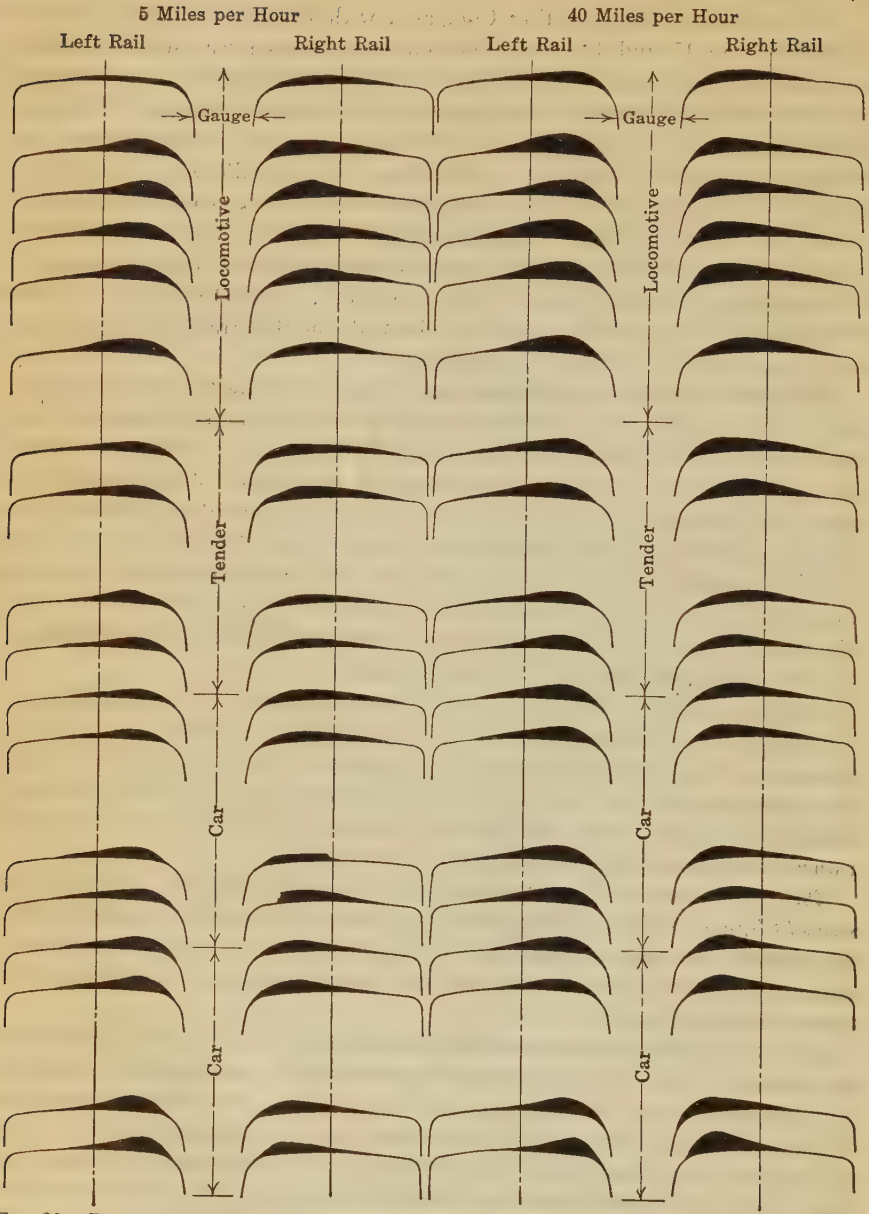


FIG. 80.—BEARING OF WHEEL ON RAIL. STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

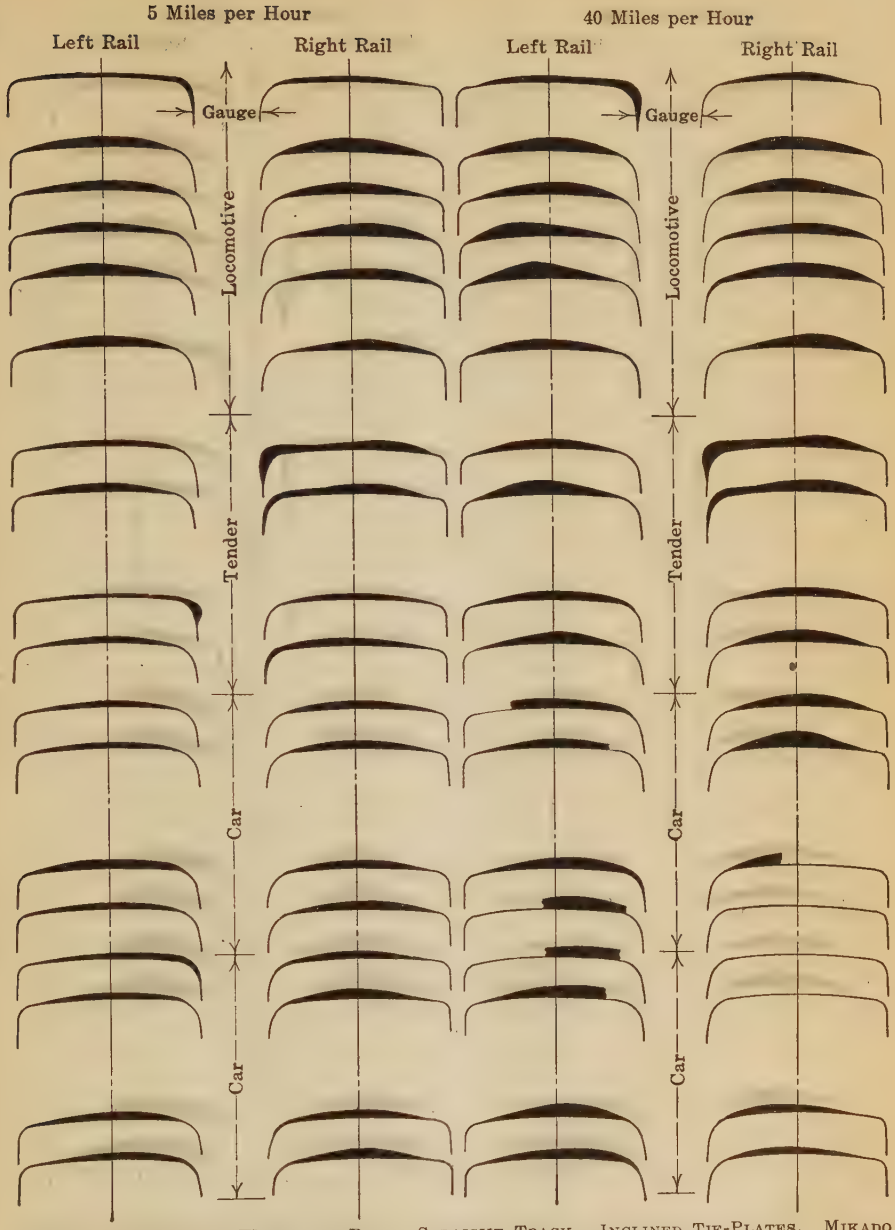


FIG. 81.—BEARING OF WHEEL ON RAIL. STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

FIG. 82.—BEARING OF WHEEL ON RAIL. STRAIGHT TRACK. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. READING COMPANY.

FIG. 83.—BEARING OF WHEEL ON RAIL. STRAIGHT TRACK. INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. READING COMPANY.

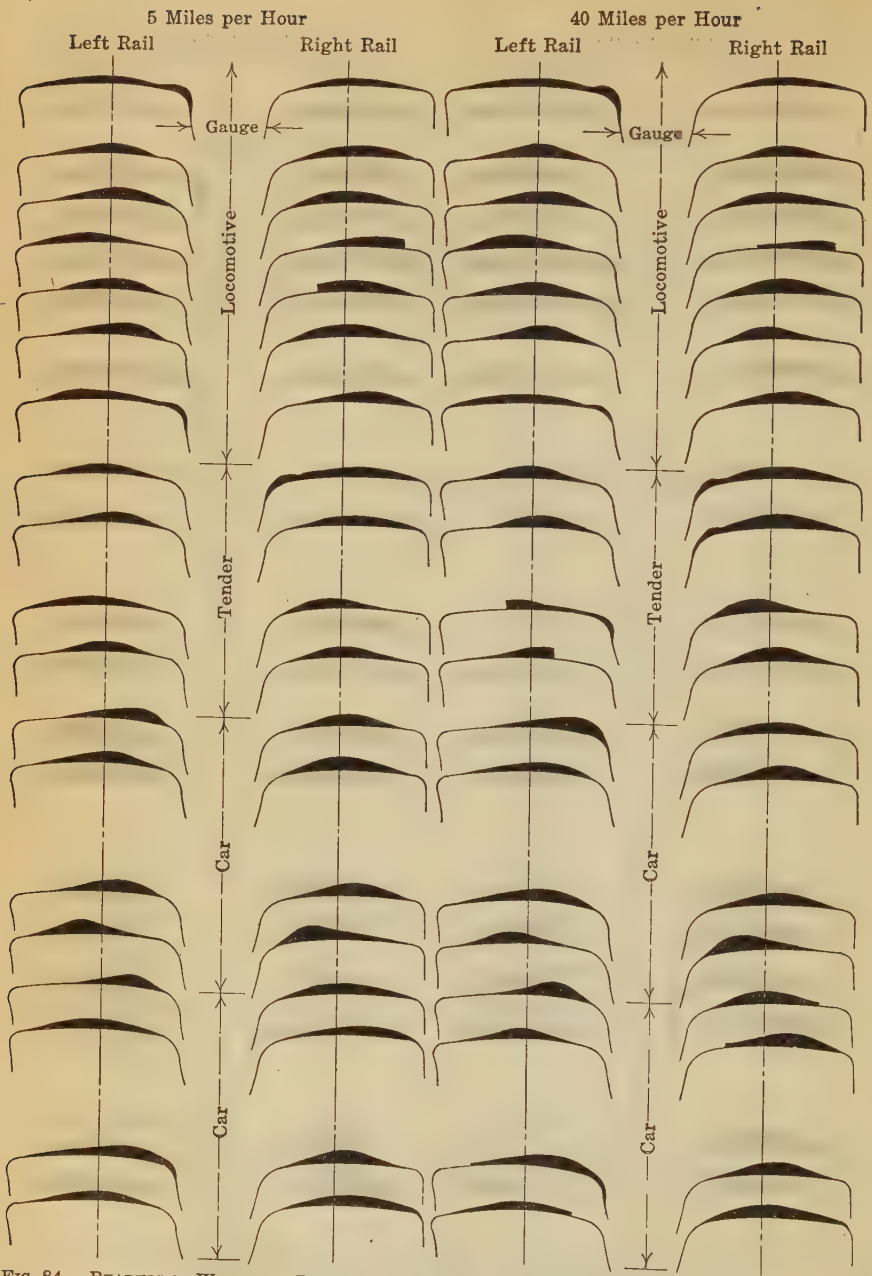


FIG. 84.—BEARING OF WHEEL ON RAIL. STRAIGHT TRACK. INCLINED TIE-PLATES. SANTA FÉ TYPE LOCOMOTIVE AND CARS. LEHIGH VALLEY RAILROAD.

influenced by the two classes of tie-plates in much the same way as the lateral bending of the rail.

It is also evident that the bearing of wheels on the rail is closer to the middle of the head of the rail when the rail is canted inwardly than when it is vertical. This result was quite marked and was common for the drivers and the wheels of the tenders and the loaded freight cars. The distribution of brightness and of the wear over the head of the rail on the track tested covered a greater width of the head for the canted rail than for the vertical rail.

The actual cant of the rail as measured in the track differed from that indicated by the design of the tie-plates, whether they were flat or inclined. This variation in the position of the rail may be expected to be attributable to other sources than the inclination of the tie-plate itself—probably, as will be seen, to the amount of eccentricity of the tie-plate.

As far as the data of the tests are conclusive, an inward inclination of the tie-plate of 1 in 20 is effective on straight track in reducing the average value of the lateral bending stresses in the rail and in securing bearing on the rail that is fairly near central.

There is no evidence that the canting of the rail has an effect on the general lateral movement of the rail or on changes in gauge, either narrowing or widening, except possibly as a slightly greater eccentricity of tie-plate may be needed in the case of the inclined tie-plate to prevent outward tilting of the rail.

It should be remembered that the average lateral bending stresses referred to as being small when the rail is canted 1 in 20 and as being greater when the rail is vertical are the averages of a number of runs. Whether or not the rail is canted there will still be the variation on either side of the average. If the average ratio of stress at outside edge to mean stress in base of rail is 1.00, there will still be a belt of values ranging from as much as a 20% greater stress at the outside edge to a 20% greater stress at the inside edge, with many occasional values as great as 30% more or less than the average, and some even greater variations. If the average ratio with a flat tie-plate were 1.10, the corresponding upper range would reach to a stress at the outside edge of base of rail 30% greater than the mean stress for the ordinary upper limit, with other still greater additional stresses. Even if the average lateral bending stress is small, it is seen that there will still be need for adequate lateral strength and stiffness in the rail. The use of tie-plates with the proper inclination will not remedy this situation.

Independently of the lateral bending of the rail just considered, as the load passes by there is a tilting of the rail inwardly or outwardly due to the greater bearing pressure on the tie under the tie-plate at one side of the rail or the other, and the consequent greater depression of the tie-plate at that end. If the load were applied to the middle of the head and were vertical, the design of the tie-plate should be symmetrical with respect to the rail. When the resultant force applied to the rail is inclined outwardly of the track, the use of a symmetrical tie-plate may result in an outward tilting of the rail

and a widening of the gauge. The use of an unsymmetrical tie-plate having the greater projection on the outside of the rail will tend to correct this condition. Too great an eccentricity in the tie-plate will result in an inward tilting of the rail. The modulus of elasticity of the wood of the tie when compressed across the grain is small in comparison with that lengthwise of the grain, say, one-fiftieth part, if the detrusion of the tie-plate due to bearing pressure is included. As a result, small differences in the distribution of pressure along the tie-plate will throw it out of level and result in a tilting of the rail and, finally, in a permanent depression of one end of the tie-plate and a cutting of the tie. What eccentricity of tie-plate will be most satisfactory is a matter to be determined by experience. The observations made on the tilting of the rail in the tests, however, may be of service in judging of what may be considered a proper value.

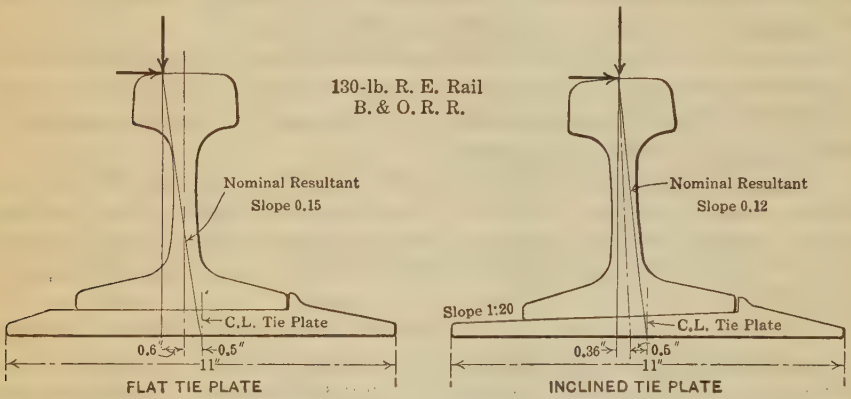


FIG. 85.—POSITION OF NOMINAL RESULTANT WITH FLAT AND INCLINED TIE-PLATES.

For the tests on all four railroads with both flat tie-plates and inclined tie-plates, the tilting of the rail as the load passed by (see Figs. 76 and 77 and discussion) was inward in all cases except for the low rail of the Richmond, Fredericksburg and Potomac Railroad laid with flat tie-plates having an eccentricity of ¼ in., which was found to have an outward cant of 1 in 30 and which tilted outwardly under load, due in all probability to the lateral component developed by the transverse inclination of the track. Almost without exception on all the track the inward inclination of the rail from the vertical was greater than that due to the form of the tie-plate. It would seem then that the increase in the cant of the rail is due to continued tilting under the passage of trains and uneven pressure on the tie. The tests made heretofore on track with symmetrical tie-plates and without tie-plates have shown a slight outward movement of the rail, and it is common experience that the gauge on such track generally widens.

It will be well also to study the position of the center of the tie-plate with respect to that of the bearing of the wheel on the rail. To bring out the position of the forces acting, Fig. 85 has been prepared, which shows the section of the 130-lb. rail of the Baltimore and Ohio Railroad with the flat tie-plate and the inclined tie-plate.

The eccentricity of the tie-plate in both cases (distance from the center of the rail to the center of the plate) is 0.5 in. The inclination of the inclined tie-plate is 1 in 20. If the vertical force is applied to the head of the rail at a point 0.6 in. inside the middle of the head in the case of the vertical rail and at the middle of the head of the canted rail, and if the amount of the lateral force in each case is such that the resultant passes through the center of the tie-plate, the real eccentricity of the force for the two cases would be 1.10 and 0.86 in., respectively, and the slope of the resultant (tangent of the angle with the vertical) 0.15 and 0.12, respectively, or 1 horizontal to 6.8 and 8.4 vertical. Such slopes are greater than would be expected from the values of the average lateral bending stresses in any of the tests found thus far. It would appear then that the resultant would fall inside the center of the tie-plate in such cases.

Table 16 gives information on the test track from which the possible eccentricity of the resultant force may be studied. Fig. 85 will help in understanding the terms used. The center of the wheel bearing with respect

TABLE 16.—Center of Wheel-Bearing, Eccentricity of Tie-Plate, Cant of Rail, Nominal Eccentricity, and Slope of Nominal Resultant on Straight Track.

(The values are given in inches, the positive direction being inward from the middle of the head of rail. The position of the center of the wheel bearing was estimated from the impressions of copper wire for the wheels of locomotives and cars when available and otherwise from the appearance of the rail and other observations. The nominal eccentricity is the horizontal distance from the middle of the length of the tie-plate to a vertical line drawn through the center of the wheel bearing. (See Fig. 85.) The so-called nominal resultant is assumed to be acting along a line joining the center of the wheel bearing and the middle of the tie-plate, and the slope given is that between this line and the vertical (tangent of the angle)).

Location.	Average cant of rail.	Center of wheel bearing.	Eccentricity of tie-plate.	Eccentricity due to cant of rail.	Nominal eccentricity.	Slope of nominal resultant.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Baltimore & Ohio Railroad:						
Flat tie-plate	1:60	0.60	0.50	0.12	1.22	0.16
Inclined tie-plate	1:16	0.00	0.50	0.45	0.95	0.13
Reading Co.:						
Flat tie-plate	1:200	0.30	0.50	0.04	0.84	0.11
Inclined tie-plate	1:16	−0.10	0.50	0.45	0.85	0.12
Lehigh Valley Railroad:						
Inclined tie-plate*	1:18	0.00	0.15	0.42	0.57	0.08
Richmond, Fredericksburg & Potomac Railroad:						
Flat tie-plate	00	0.30	0.25	0.00	0.55	0.09
	−1:30	0.70	0.50	−0.21	0.74	0.12
Inclined tie-plate	1:20	0.10	0.50	0.33	0.93	0.14

* Worn rail.

to the middle of the head of the rail, as judged from the impressions of copper wire and the wear and brightness of the rail, the eccentricity of the tie-plate (distance from the center of base of rail to the mid-point of the tie-plate), and the eccentricity due to cant of rail are given. The position of the resultant of the gravity load and the lateral force on the rail will be unknown. A force passing through the center of the wheel bearing and the mid-point of the tie-plate will be called the nominal resultant. This nominal resultant will give equilibrium only in case the average lateral force acting upon the rail is of such magnitude as to make the resultant pass through the mid-point of the tie-plate. If the nominal resultant were the actual resultant, the pressure on the tie-plate would be uniformly distributed over it. The slope of this nominal resultant and the corresponding nominal eccentricity of such a line are given in Table 16. A study of the eccentricities and the tilting of the rail under load may throw light on the position of the actual resultant.

In comparing the nominal eccentricity given in Table 16, it should be remembered that the flat tie-plate on the Baltimore and Ohio Railroad had only recently been laid and, therefore, that no change in cant had occurred with these tie-plates; no direct conclusion can be drawn concerning the nominal eccentricity, 1.22 in., except as may be judged from the tilt of the rail under load. The rail on the test track on the Lehigh Valley Railroad was considerably worn and, therefore, the increase in cant in that rail may not have been caused by conditions existing at the time of the test. As the track of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates, has one rail 2 in. lower than the other, the cant of the two rails must have been influenced by the transverse inclination of the track. However, it is seen that in nearly every case the cant of the rail inward had been increased by the traffic from that at which presumably it was laid.

From the data available it may be concluded that for both flat tie-plates and inclined tie-plates the tie-plates should be unsymmetrical. It may be concluded also that with the rails and tie-plates used, an eccentricity of 0.5 in. is too great. Possibly $\frac{1}{4}$ in. may be nearer the proper amount. It would seem better to state the eccentricity of the combination in terms of the position of the wheel bearing, the cant of the rail, and the eccentricity of the tie-plate itself. For the size of rail and tie-plate used in the track tested, it may be judged that 0.75 in. is a proper value for the nominal eccentricity. With the 130-lb. rail this is about one-tenth the height of the rail and tie-plate. With a cant of 1 in 20 and a central wheel bearing on the rail, an eccentricity of tie-plate of $\frac{1}{4}$ to $\frac{3}{8}$ in. would result. Whether this eccentricity of tie-plate will give proper conditions may best be told by experience.

The position of the resultant gravity load and the lateral force referred to has been assumed to be that for the average of a number of applications of load. Due to variations in the lateral pressure on the rail, it will be seen that individual runs will develop resultants intersecting the bottom of the tie-plate at one side or the other of the average position. Such positions of the resultant will give uneven distributions of bearing pressure between tie-plate and tie, a higher intensity of pressure at one edge, and a lower pressure

at the other edge. The increase in intensity will depend not only on the position of the resultant, but also on the length of the tie-plate. As an illustration of change in intensity, consider that the main belt of ratios of stress at edge to mean stress in base of rail (discussed in Article 23, "Lateral Bending Stresses in Straight Track and Ratios of Stress at Outside Edge to Mean Stress in Base of Rail") extends 0.30 on each side of the average ratio. Assuming that the lateral force producing the lateral bending bears the same relation to the vertical load as the lateral bending moment bears to the vertical bending moment developed at the point (an assumption that seems fairly reasonable even though no conclusive information is available), and considering that the section modulus, $\frac{I}{c}$, of the rail about the vertical axis is one-fifth that about the horizontal axis, it is found that the addition of 0.30 to the stress ratio means an addition to the lateral force on the rail equal to 0.06 of the vertical force. This means for the rail and tie-plate shown in Fig. 85 that the resultant will intersect the bottom of the tie-plate 0.06 times 7½ in., a distance of, say, 0.45 in. from the average position. Assuming that the average position of the resultant passes through the middle of the length of the tie-plate and that the tie-plate is 11 in. long, the relative eccentricity of the plate is $\frac{0.45}{11}$. Using Equation (34), it is seen that the intensity of stress at the edge is about 25% greater or less than the average pressure on the tie. A longer tie-plate will give a smaller variation from the average and a shorter tie-plate a greater variation. Whether such a variation is or is not excessive will depend, of course, on whether the average bearing pressure is sufficiently small for the kind of wood in the tie.

Both the matter of low average pressure of plate on tie and that of a sufficiently small variation in intensity of pressure for the more extreme variations in lateral forces against the rail bring emphasis to the need of adequate length of tie-plates. It is apparent that the position of the resultant at the tie is a function of the height of the rail and thickness of the tie-plate, and granted a sufficiently large bearing area, the desirable length of tie-plate may be considered a function of this total height. The dimension, 11 in., is about one and one-half times the height of the rail and the thickness of the tie-plate. Whether this tie-plate is sufficiently long for the loads carried is a matter for experience to determine, but it is evident that long tie-plates and large bearing areas have important advantages.

To recapitulate the discussion of the tests on straight track: The rails laid with an inward cant of 1 in 20 or thereabouts in general show a smaller average lateral bending than those laid in a vertical position. The canted rail on all the railroads shows a bearing between wheel and rail for both locomotive and cars that is close to the middle of the head of the rail, while with the vertical rail the bearing is well on the inside, about 0.5 in. from the middle of the head. The distribution of brightness and of wear extends over a greater width of the head for the canted rail than for the vertical rail. The results indicate an advantage in canting the rail by giving a more

nearly central bearing and a better distribution of wear on the rail and in reducing the average lateral bending stresses in the rail. The indications from the tests are that an inward cant of 1 in 20 is effective and better than a smaller one.

In this connection, it should be noted that the standard coning of the wheels of the locomotive and cars is 1 in 20, except for the Baltimore and Ohio Railroad, for which 1 in 13 is standard. In considering the results on the Baltimore and Ohio Railroad, it should be borne in mind that the average cant of the rails on the test sections of the straight track averaged 1 in 16.

Although the inward canting of the rail reduces the average of the lateral bending stresses, it has no apparent effect on the variation in the lateral bending stresses. There will still be lateral bending stresses ranging to 20 and 30% of the vertical bending stress, with occasional values still greater, the stress in an edge of the base of the rail being increased to this extent over the mean stress in base of rail. The need for adequate lateral strength and stiffness in the rail still remains.

The position of the wheel bearing on the vertical rail, and that on the canted rail, together with the cant of the rail, indicates that in both cases the resultant of the vertical and lateral forces on the average slope away from the vertical. The general inward tilting of the rail in the tests is evidence that for both the flat tie-plates and the inclined tie-plates used the eccentricity of the tie-plates (generally 0.5 in.) is too great, making, as it does, with the eccentricity of bearing on the vertical rail or the cant of the canted rail, a total eccentricity of 0.85 to 1.20 in. on the 130-lb. rail. A slope of 1 in 10 from the center of bearing of wheel on rail to the mid-point of the base of the tie-plate, or a little less, would appear a satisfactory value to be tested by experience.

Variations in the lateral force applied to the rail will result in variable intensities of bearing pressure on the tie, which may be decreased by increasing the length of the tie-plate. Granting sufficient tie-bearing area, the length of the tie-plate may well be expressed in terms of the height of rail and thickness of tie-plate. The length of the tie-plates used with the 130-lb. rail was about one and one-half times the vertical dimension referred to. Adequacy of bearing areas and of length of plate is a very important matter.

27.—Stresses in Rail on Curved Track.—In Figs. 86 to 90 are plotted the stresses found at the inside edge and the outside edge of the base of the outer rail and the inner rail under the wheels of the locomotive and loaded freight cars on the two 7° curves of the Baltimore and Ohio Railroad, the 10° curve of the Lehigh Valley Railroad, and the 2½° and the 5° curves of the Richmond, Fredericksburg and Potomac Railroad. The stresses given are average values for, say, ten runs with a possible of forty readable observations at each edge of each rail. The observations were so distributed through the revolution of the drivers that the effect of counterbalance was entirely masked. An effect of counterbalance may be expected, of course, which will give an increased stress at certain points in the revolution of the driver.

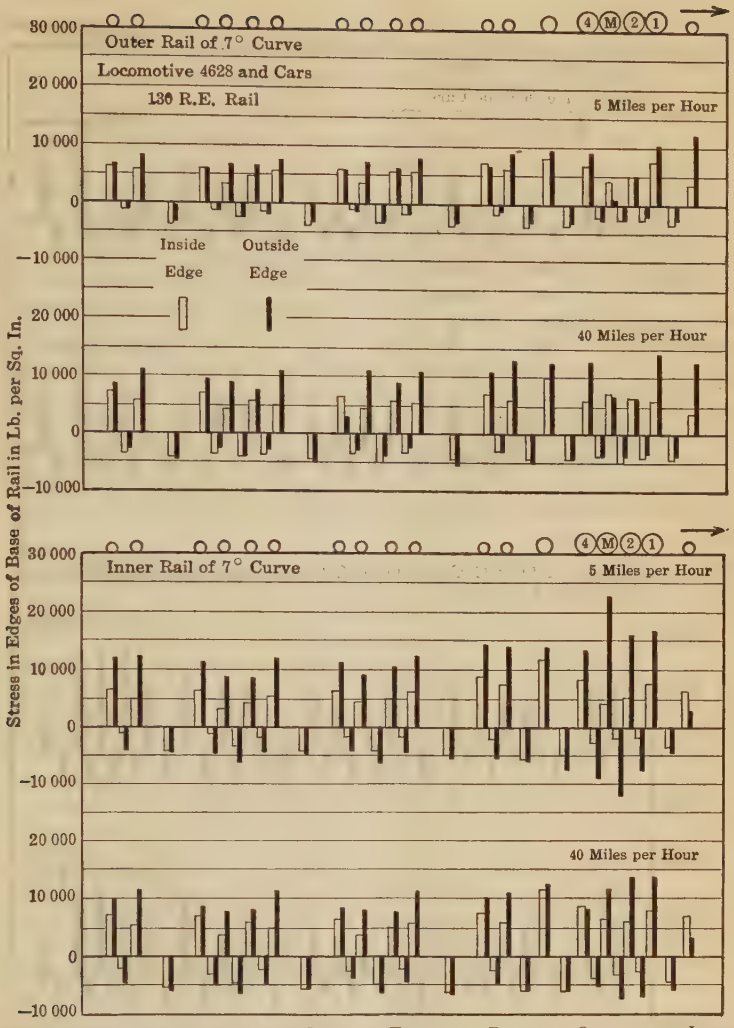


FIG. 86.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF BASE OF OUTER AND INNER RAILS OF THE 7° CURVE WITH FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

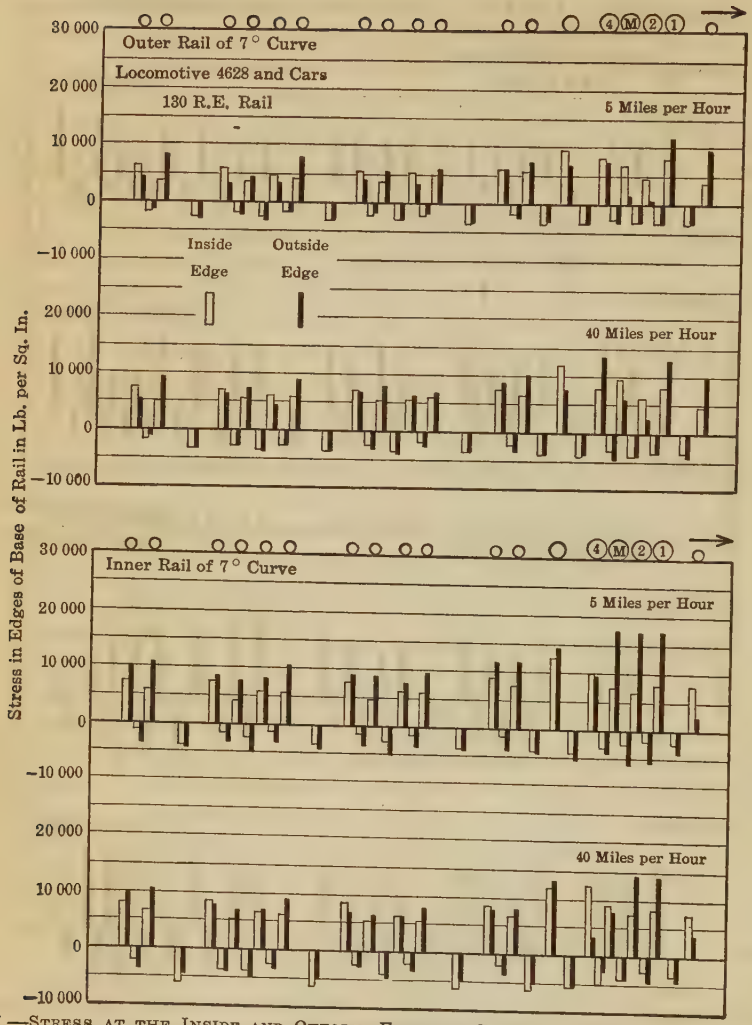


FIG. 87.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF BASE OF OUTER AND INNER RAILS OF THE 7° CURVE WITH INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

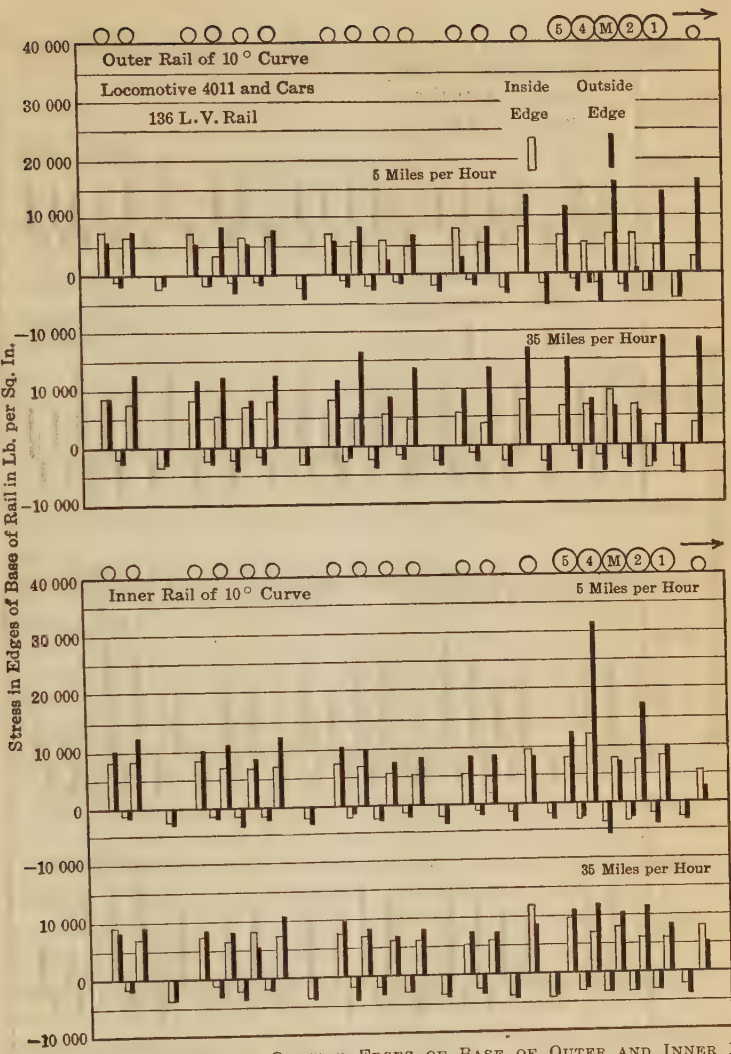


FIG. 88.—STRESS AT INSIDE AND OUTSIDE EDGES OF BASE OF OUTER AND INNER RAILS OF THE 10° CURVE WITH INCLINED TIE-PLATES. SANTA FÉ TYPE LOCOMOTIVE AND CARS, LEHIGH VALLEY RAILROAD.

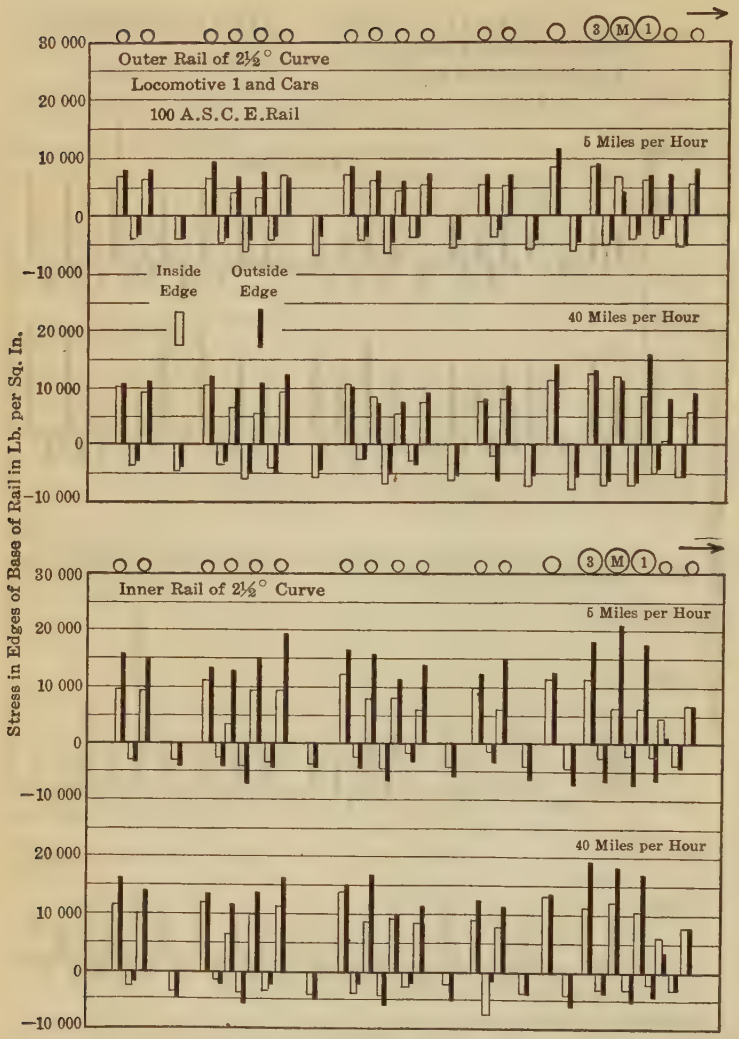


FIG. 89.—STRESS AT INSIDE AND OUTSIDE EDGES OF BASE OF OUTER AND INNER RAILS OF THE 2½° CURVE WITH FLAT TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

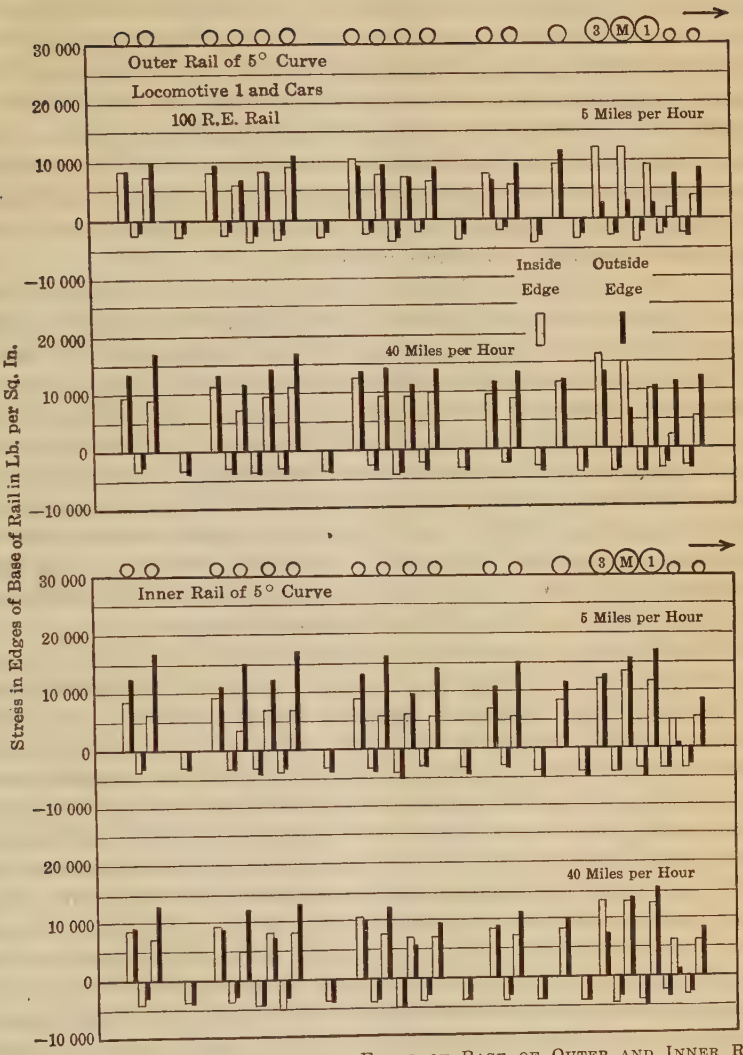


FIG. 90.—STRESS AT INSIDE AND OUTSIDE EDGES OF BASE OF OUTER AND INNER RAILS OF THE 5° CURVE WITH INCLINED TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS, RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

On the 7° curve of the Baltimore and Ohio Railroad laid with flat tie-plates, it is seen from Fig. 86 that the highest stress in the outer rail under the wheels of the Mikado type locomotive at 5 miles per hour is 12 000 lb. per sq. in., found in the outside edge under the front truck wheel, and 10 000 lb. per sq. in. under the first driver and trailer. For the inner rail (Fig. 86), the highest stress is 23 000 lb. per sq. in. at the outside edge under the third driver, with 4 500 lb. per sq. in. at the inside edge. At 40 miles per hour, the stresses in the outer rail have increased somewhat and those in the inner rail under the third driver have dropped to 12 000 lb. per sq. in., a marked change. For the cars (wheel loads averaging about 18 000 lb.), the highest stresses in the outer rail ran to 11 000 lb. per sq. in. at the outside edge of the outer rail at 40 miles per hour and to 12 500 lb. per sq. in. at the outside edge of the inner rail at 5 miles per hour.

On the 7° curve of the Baltimore and Ohio Railroad laid with inclined tie-plates (see Fig. 87), the greatest stress in the outer rail at 5 miles per hour was 9 500 lb. per sq. in. at the outside edge under the front truck wheel and 11 000 lb. per sq. in. under the first driver, and at 40 miles per hour, 12 500 lb. per sq. in. under the first and fourth drivers. In the inner rail the stresses under the first, second, and third drivers at 5 miles per hour were 17 500 lb. per sq. in. at the outside edge, decreasing to 14 000 lb. per sq. in. or less at 40 miles per hour. For the loaded cars the greater stresses in the outer rail ranged from 7 000 to 9 000 lb. per sq. in. at 40 miles per hour and in the inner rail up to 10 000 lb. per sq. in. at 5 miles per hour. It is seen that the values of the higher stresses were noticeably greater in both rails when laid with flat tie-plates than when laid with inclined tie-plates.

On the 10° curve of the Lehigh Valley Railroad, laid with inclined tie-plates (see Fig. 88), the greatest stresses in the outer rail under the wheels of the Santa Fé type locomotive ran to 16 000 lb. per sq. in. at 5 miles per hour and to 19 000 lb. per sq. in. at 35 miles per hour, with considerable variation for the different drivers. In the inner rail the greatest stress was 31 500 lb. per sq. in. at the outside edge under the fourth driver at 5 miles per hour, with 12 000 lb. per sq. in. at the inside edge. These stresses indicate not only high lateral bending force at this driver, but show that the vertical load transmitted by this driver under the condition was far greater than the nominal load, a result that agrees with tests given in the third progress report. At the speed of 5 miles per hour, the head of the inner rail moved outwardly under the fourth driver 0.21 in., a considerable movement for so stiff a rail, especially as it occurred in a short length. At 35 miles per hour the stresses under this driver decreased to 12 000 and 7 000 lb. per sq. in. at the two edges of the rail, and there seemed to be less difficulty in its traversing the curve. The worn section of the inner rail probably contributed to the increase in mean stress in base of rail by perhaps 5%, but the effect on lateral bending was, of course, much smaller. The loaded cars (wheel load of 24 000 lb.) gave stresses in the outside edge of the outer rail up to 8 000 lb. per sq. in. at 5 miles per hour and to 12 000 lb. per sq. in. at 35 miles per hour, with a stress of 16 500 lb. per sq. in. under one wheel, the stresses being

generally higher under the first wheel of a truck. In the inner rail, the stress at the outside edge was generally the higher, and ranged to 12 500 lb. per sq. in. at 5 miles per hour and to 10 000 and 11 000 lb. per sq. in. at 35 miles per hour. In making comparison it should be borne in mind that the wheel load of these cars was 33% greater than those used on the Baltimore and Ohio Railroad.

On the 2½° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates (see Fig. 89), the stresses in the outer rail under the wheels of the locomotive and cars at 5 miles per hour are less than on a straight track, becoming a little greater at 40 miles per hour. In the inner rail the stresses at the outside edge under the drivers range from 17 000 to 22 000 lb. per sq. in., becoming less at 40 miles per hour. For the loaded cars the stresses in the inner rail are considerably higher than on straight track. This rail was canted outwardly, presumably by the action of traffic.

On the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with inclined tie-plates (see Fig. 90), the stresses in the outer rail were highest at the inside edge under the second and third drivers, 12 000 lb. per sq. in. at 5 miles per hour, and 15 500 and 16 500 lb. per sq. in., respectively, at 40 miles per hour. In the inner rail the stress at the outside edge under the first driver at 5 miles per hour was 17 000 lb. per sq. in., a smaller value than that found on the 2½° curve; it became somewhat less at 40 miles per hour. For the cars (wheel load of 25 000 lb.), the greatest stress in the outer rail was less than 10 000 lb. per sq. in. at 5 miles per hour and ran to 16 500 lb. per sq. in. at 40 miles per hour, the latter being as high a stress as was found under the wheels of the locomotive. In the inner rail the stress under the cars ran to 17 000 lb. per sq. in. at the outside edge at 5 miles per hour and to 13 000 lb. per sq. in. at 40 miles per hour, the greatest stress usually being under the first wheel of the truck.

Above and below the average stress at each edge of the base of rail here reported, there was the same range or belt of stresses as has been described in other tests.

28.—Vertical Bending Stresses on Curved Track.—As before, the mean stress in base of rail is taken as the vertical bending stress, or better, the stress in a line normal to the transverse inclination of the track. For the different speeds and the different cants the mean stress may not represent the normal bending stress exactly, but usually for the conditions of the tests it will not differ more than 1% from the correct results. The vertical bending stresses (the averages for all the runs) for the five curves are recorded in Figs. 91 to 95. For convenience, the curves are shown as curving to the right although all but the 2½° curve to the left with respect to the direction of the runs.

On the 7° curve of the Baltimore and Ohio Railroad laid with flat tie-plates, the highest vertical bending stress in the outer rail is under the first driver of the Mikado type locomotive, 10 000 lb. per sq. in. at 40 miles per hour; in the inner rail the highest stresses at 5 miles per hour are 13 600 lb. per sq. in. under the third driver and 13 200 lb. per sq. in. under the trailer,

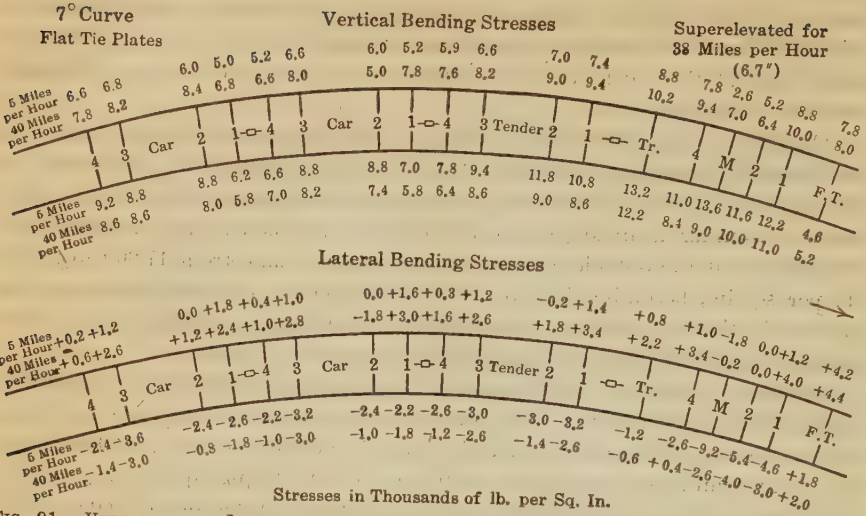


FIG. 91.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 7° CURVE WITH FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

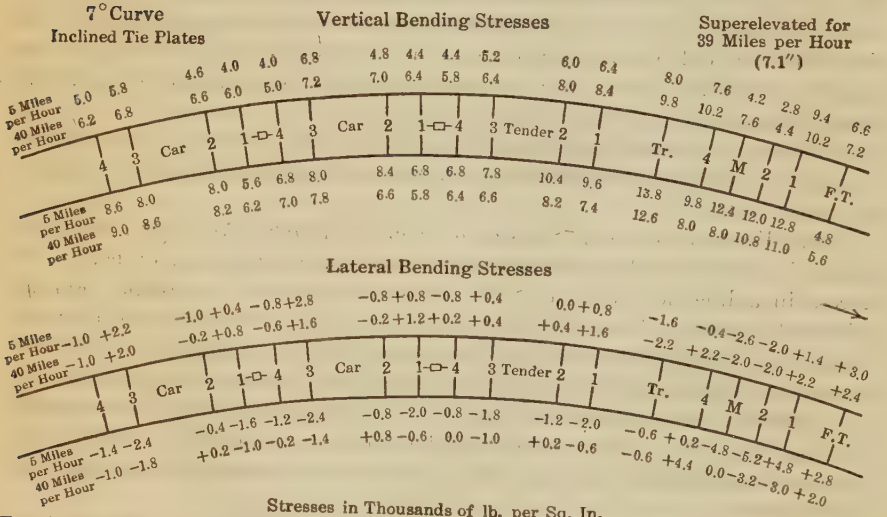


FIG. 92.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 7° CURVE WITH INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

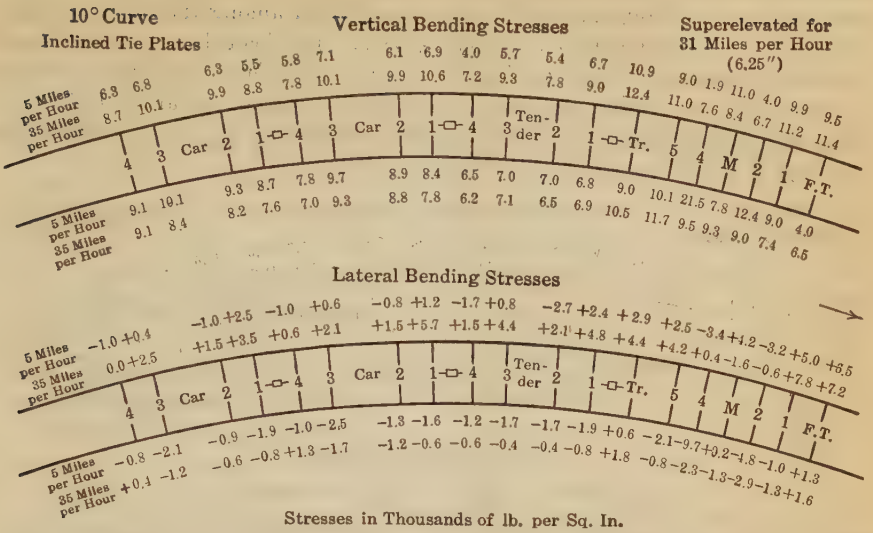


FIG. 93.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE WITH INCLINED TIE-PLATES. SANTA FÉ TYPE LOCOMOTIVE AND CARS. LEHIGH VALLEY RAILROAD.

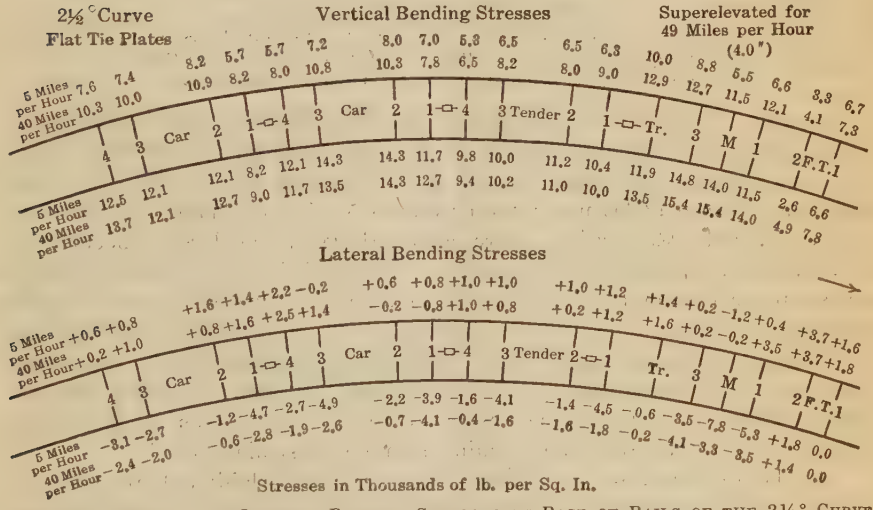


FIG. 94.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 2½° CURVE WITH FLAT TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS. RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

and at 40 miles per hour 11 000 and 12 200 lb. per sq. in., respectively, under the same wheels. The stresses in the inner rail at 5 miles per hour do not give evidence of highly uneven distribution of load among the drivers at low speeds, as has been reported in previous tests. The loaded cars (wheel loads of 18 000 lb.) gave vertical bending stresses in the outer rail as great as 6 800 lb. per sq. in. at 5 miles per hour and 8 400 lb. per sq. in. at 40 miles per hour, and in the inner rail values as great as 9 200 lb. sq. in. at 5 miles per hour and 8 600 lb. per sq. in. at 40 miles per hour.

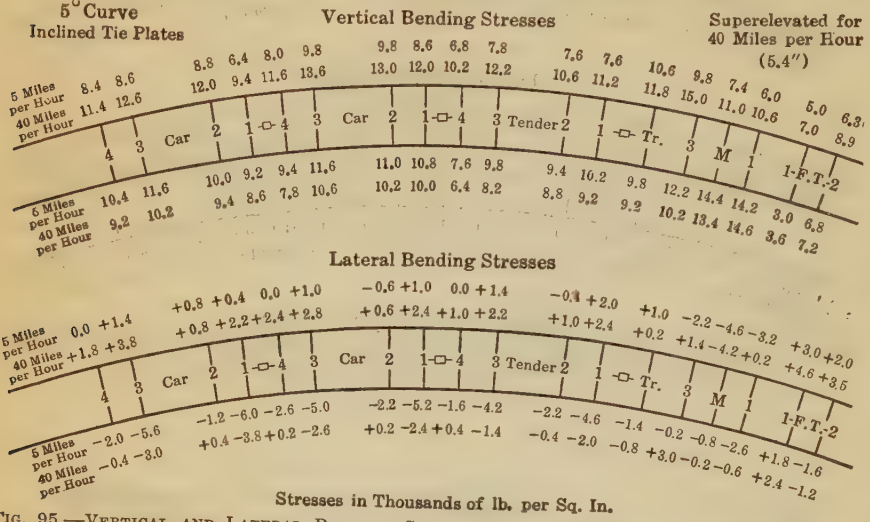


FIG. 95.—VERTICAL AND LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 5° CURVE WITH INCLINED TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS. RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

On the 7° curve of the Baltimore and Ohio Railroad with inclined tie-plates, the vertical bending stresses in the outer rail ran to 10 200 lb. per sq. in. under the first and fourth drivers at 40 miles per hour, and in the inner rail from 9 800 to 12 800 under the drivers at 5 miles per hour, and 13 800 lb. per sq. in. under the trailer. Here, again, there was no large concentration of load by one driver on the inner rail. The loaded cars developed 6 800 lb. per sq. in. in the outer rail at 5 miles per hour and 7 200 lb. per sq. in. at 40 miles per hour; in the inner rail, 8 000 lb. per sq. in. at 5 miles per hour and 9 000 at 40 miles per hour. These values do not differ greatly from those found on the 7° curve laid with flat tie-plates. It was also found that the average of the vertical bending stresses in the two rails under all the drivers was no greater than the corresponding average on straight track.

On the 10° curve of the Lehigh Valley Railroad, laid with inclined tie-plates, the highest vertical bending stress in the outer rail at 35 miles per hour was 12 400 lb. per sq. in. under the trailer of the Sante Fé type locomotive and 11 000 lb. per sq. in. under the front truck wheel and the first and fifth drivers. In the inner rail the greatest stress was 21 500 lb. per sq. in. under the fourth driver at 5 miles per hour (accompanied by a stress of 1 900 lb.

per sq. in. in the outer rail), showing a severe concentration of load on this inner driver, which must result in injury to the inner rail, results which have been found elsewhere with this type of locomotive when the third driver was flangeless. At the speed of 35 miles per hour the stress in the inner rail under the fourth driver decreased to 9 500 lb. per sq. in. For the loaded cars the stress in the outer rail ran to 10 600 lb. per sq. in. at 35 miles per hour and that in the inner rail to 10 100 lb. per sq. in. at 5 miles per hour.

On the 2½° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates, the higher vertical bending stresses in the outer rail at 5 miles per hour were 13 000 lb. per sq. in. under the wheels of the locomotive and 11 000 lb. per sq. in. under the cars. In the inner rail at the same speed, the greatest stress was 14 800 lb. per sq. in. under the locomotive and 14 300 lb. per sq. in. under the cars. Little change in stress in the inner rail with change in speed was found under locomotive or cars, but an increase with increase in speed was found in the outer rail.

On the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with inclined tie-plates, the vertical bending stress in the outer rail under the drivers of the locomotive ran to 15 000 lb. per sq. in. at 40 miles per hour and in the inner rail to 14 500 lb. per sq. in. at both speeds. For the cars, the stress in the outer rail was 9 400 to 13 600 lb. per sq. in at 40 miles per hour and in the inner rail, 9 200 to 11 600 lb. per sq. in. at 5 miles per hour. The stresses found in the 5° curve were not greater than those in the 2½° curve.

29.—Lateral Bending Stresses on Curved Track.—In Figs. 91 to 95 are recorded the average values of the lateral bending stresses in base of rail under each wheel of the locomotive and cars for the five curves at the two speeds. The lateral bending stress given is the difference between the stress at the outside edge and the mean stress in base of rail (half of the difference between the stresses measured at the two edges). When the positive sign is used, the stress tends to increase the curvature of the rail; when negative, it tends to straighten the rail. A positive bending stress, then, indicates a tendency to deflect the outer rail outwardly of the track and the inner rail inwardly of the track. This flexural action exists in addition to any general outward or inward movement of the rail that may occur.

In the two 7° curves of the Baltimore and Ohio Railroad, it is seen from the positive stresses in the outer rail under the front truck wheel and first driver that the guiding force making the Mikado type locomotive traverse the curve is applied to the outer rail by these two wheels. A lateral pull on the inner rail is also exerted by the inner wheel of the front truck. The first outer wheel of a truck of the tender and cars also gives positive bending in the outer rail. In the inner rail, except under the wheel of the front truck, the bending almost without exception is negative. The lateral bending stress in the outer rail under the front truck wheel and first driver, at a speed of 40 miles per hour, developed by the guiding action, was 4 000 lb. per sq. in. in the 7° curve laid with flat tie-plates and less than 3 000 lb. per sq. in. in the curve laid with inclined tie-plates, which are relatively small lateral bending

stresses. The outward thrust of the drivers on the inner rail develops lateral bending stresses of 5 000 and 9 000 lb. per sq. in. in the track laid with flat tie-plates and 5 000 lb. per sq. in. in that laid with inclined tie-plates. For both outer and inner rail the lateral bending stresses under the wheels of the cars are less in the track laid with inclined tie-plates than in that laid with flat tie-plates. It is seen that, with all other conditions the same, the lateral bending stresses in both rails were smaller for the locomotive and cars on the curve with the rail canted. Except for one driver on one curve, the lateral bending stresses were well distributed among the wheels, and the train traversed the curve seemingly without difficulty.

The lateral bending stresses in the outer rail of the 10° curve of the Lehigh Valley Railroad under the front truck wheel and first driver of the Santa Fé type locomotive, at 35 miles per hour, were 7 200 and 7 800 lb. per sq. in., respectively, values which are about two-thirds of the corresponding vertical bending stress. At these wheels the head of the rail moved outward 0.12 in. at the speed of 35 miles per hour. In the inner rail under the fourth driver, at a speed of 5 miles per hour, a high lateral bending stress was found, a characteristic of this type of locomotive when the third driver is flangeless, 9 700 lb. per sq. in., tending to strengthen the rail and indicating a strong outward push on it at this point, the rail moving outward 0.2 in. This lateral bending stress was nearly one-half as much as the extremely high vertical bending stress found under this driver. At 35 miles per hour the lateral bending stress was only 2 300 lb. per sq. in., about one-fourth the vertical bending stress found at this speed. The lateral bending stresses in the outer rail under the first wheel of a truck of the tender and cars indicate that these wheels guide the truck around the curve. There is considerable variation in the lateral bending stresses under these wheels, the range being from 2 000 to 5 700 lb. per sq. in., at 35 miles per hour, all being high with respect to the vertical bending stresses at the same points. The lateral bending stresses in the inner rail were low at both speeds and generally indicated negative bending. No comparison of lateral bending stresses with rail laid with flat tie-plates can be made.

On the 2½° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with flat tie-plates, the outer truck wheels give positive bending at both speeds, as does also the first driver of the Pacific type locomotive at 40 miles per hour, the lateral bending stresses being less than 4 000 lb. per sq. in. In the inner rail, there is a lateral bending stress of 7 800 lb. per sq. in. under the main driver, at 5 miles per hour, more than one-half as great as the vertical bending stress. Although the stress is decreased to 3 300 lb. per sq. in., at 40 miles per hour, it is evident that this wheel exercises a strong outward pressure against the inner rail and thus may have some influence in producing the outward canting of this rail. Under the cars the lateral bending stresses in the outer rail are small, but there are indications that the trucks were tending to run obliquely toward the inner rail. The lateral bending in the inner rail under the cars is inward, but the stresses under the front wheel of a truck are high at 5 miles per hour, reaching 5 000 lb. per sq. in.; at 40

miles per hour the stresses at these points are small, but the values are larger than the corresponding ones in the outer rail. Doubtless, part of this was due to the condition of the track at the inner rail.

On the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with inclined tie-plates, the outward lateral bending in the outer rail was under the front truck wheels, and evidently these wheels gave the guiding action. The lateral bending stresses were 3 500 and 4 600 lb. per sq. in., respectively, at 40 miles per hour. The lateral bending stress in the inner rail under the middle driver is very small. The lateral bending stresses in the outer rail under the tender and cars are greater for the first wheel of each truck, usually about 2 500 lb. per sq. in., at 40 miles per hour. The lateral bending stresses in the inner rail at 5 miles per hour were relatively high, ranging to 6 000 lb. per sq. in. under the first wheels of a truck. At 40 miles per hour the corresponding stresses were about one-half as much.

In Fig. 96 are plotted the values of the lateral bending moments developed in the outer and inner rail at and between wheels on the two 7° curves of the Baltimore and Ohio Railroad. The values were calculated from the average lateral bending stresses and the section modulus, $\frac{I}{c}$, of the rail about a vertical axis. They represent the tendency of the wheels of the Mikado type locomotive and cars to give lateral distortion to the rail and track.

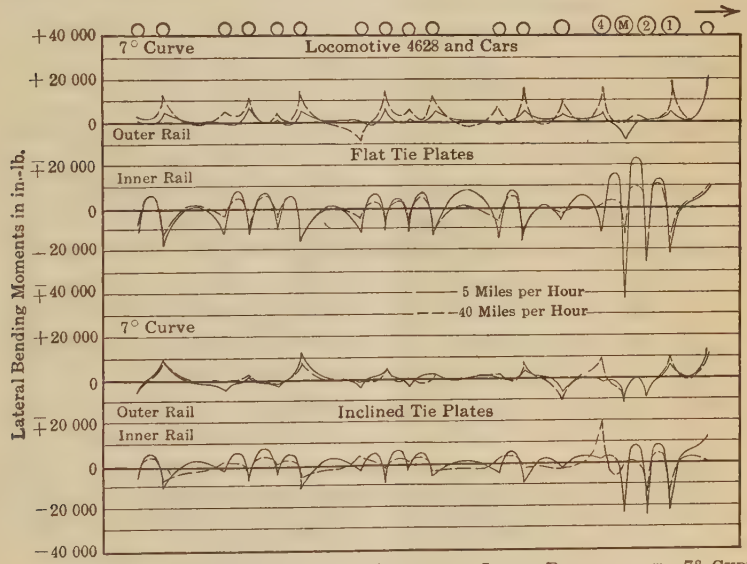


FIG. 96.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF THE 7° CURVES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

In Fig. 97, the individual lateral bending stresses at the several instruments are plotted for a speed of 40 miles per hour for the runs on the 7° curve of the Baltimore and Ohio Railroad, laid with inclined tie-plates, values under the wheels of the locomotive and of the first car being shown. It is seen

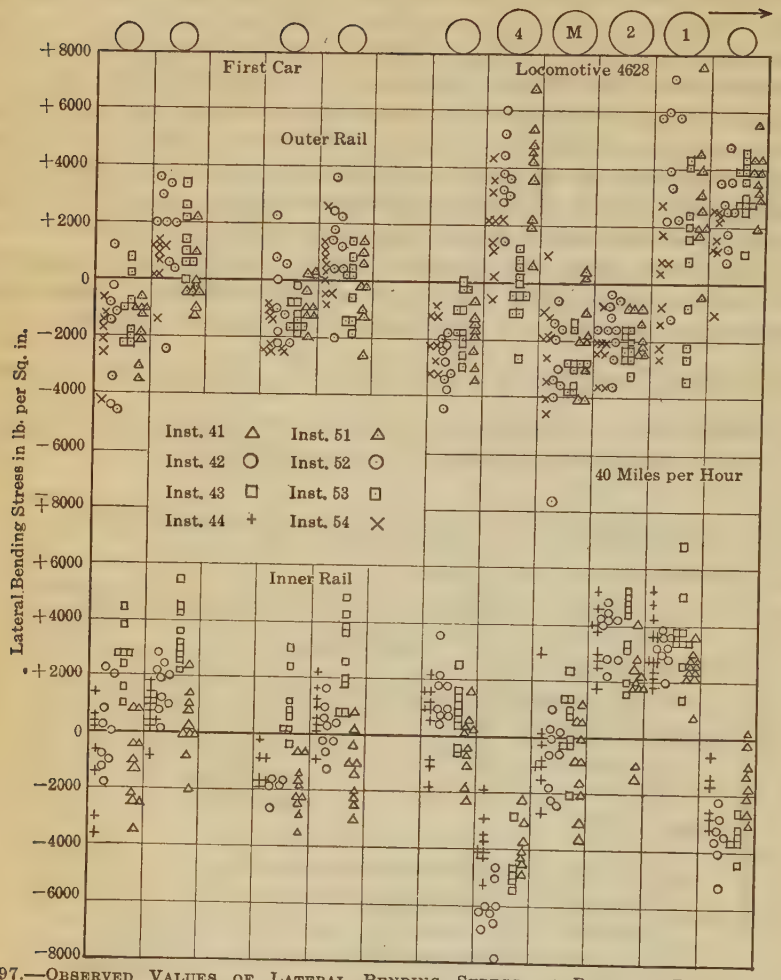


FIG. 97.—OBSERVED VALUES OF LATERAL BENDING STRESS IN BASE OF RAILS OF THE 7° CURVE WITH INCLINED TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS, BALTIMORE AND OHIO RAILROAD.

that there is a range of values for each instrument, one point on the track giving different values from another, yet the variation being much the same for different instruments and different wheels. The individual drivers give quite different results. It is evident that different runs give different results, and that the variation in values is due both to differences in the action of the train and to difference in the track from point to point.

30.—Movement of the Rail on Curved Track.—In Fig. 98 are given the measurements of tilting of the rail on the two 7° curves of the Baltimore and Ohio Railroad under the action of the locomotive and cars, and also those on the 10° curve of the Lehigh Valley Railroad and the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, the tilting of the head of the rail with respect to the base being measured as already described for the tests on straight track. The values given are the averages of two or three runs at the same point.

It is seen that the outer rail tilts outwardly very little under the wheel of the locomotive that guides the driver group. It is also seen that the inner rail generally tilts outwardly under one or more of the intermediate drivers. In general, for the other wheels of the locomotive and the wheels of the tender and cars, not only for the tests shown in the diagrams but also for the other tests, both the outer rail and the inner rail tilt inwardly and the inward tilting is about the same in the outer rail for both speeds and is greater in the inner rail at the higher speed. The tilting movement is usually less than 0.05 in., but ranges up to 0.10 in. The nature and amount of the tilting on two 1° curves of the Reading Company on which measurements of movement of rail were taken, one laid with flat tie-plates and one with inclined tie-plates, were much the same as for the other curves referred to, the action under the locomotive wheels being typical of the movement on curves.

There were some exceptions to the statements just made. The inner rail of the 10° curve of the Lehigh Valley Railroad generally tilted outwardly, markedly so under the drivers of the locomotive. It will be recalled that this inner rail was a worn rail relaid from the outer rail of the curve. The outer rail of the 2½° curve of the Richmond, Fredericksburg and Potomac Railroad tilted outwardly under both the locomotive and cars and the tilt of the inner rail under the tender and cars was slight, sometimes inwardly and sometimes outwardly. Likewise, the tilt of the inner rail of the 5° curve of the Richmond, Fredericksburg and Potomac Railroad was slight and variable in direction.

In Fig. 99 are given the measurements of the lateral movement of the head of the rail for the same four curves. An outward movement of the outer rail under the wheels which act to guide the driver group, is shown, and generally an inward movement under the other wheels. For the inner rail the movement is generally inward, except for the intermediate drivers which usually give an outward movement. The amount of the lateral movement of the head of the inner rail ranges from — 0.15 in. to + 0.20 in. The highest lateral movement of the inner rail occurred under the fourth driver of the Santa Fé type locomotive, a movement that took place almost entirely between the third

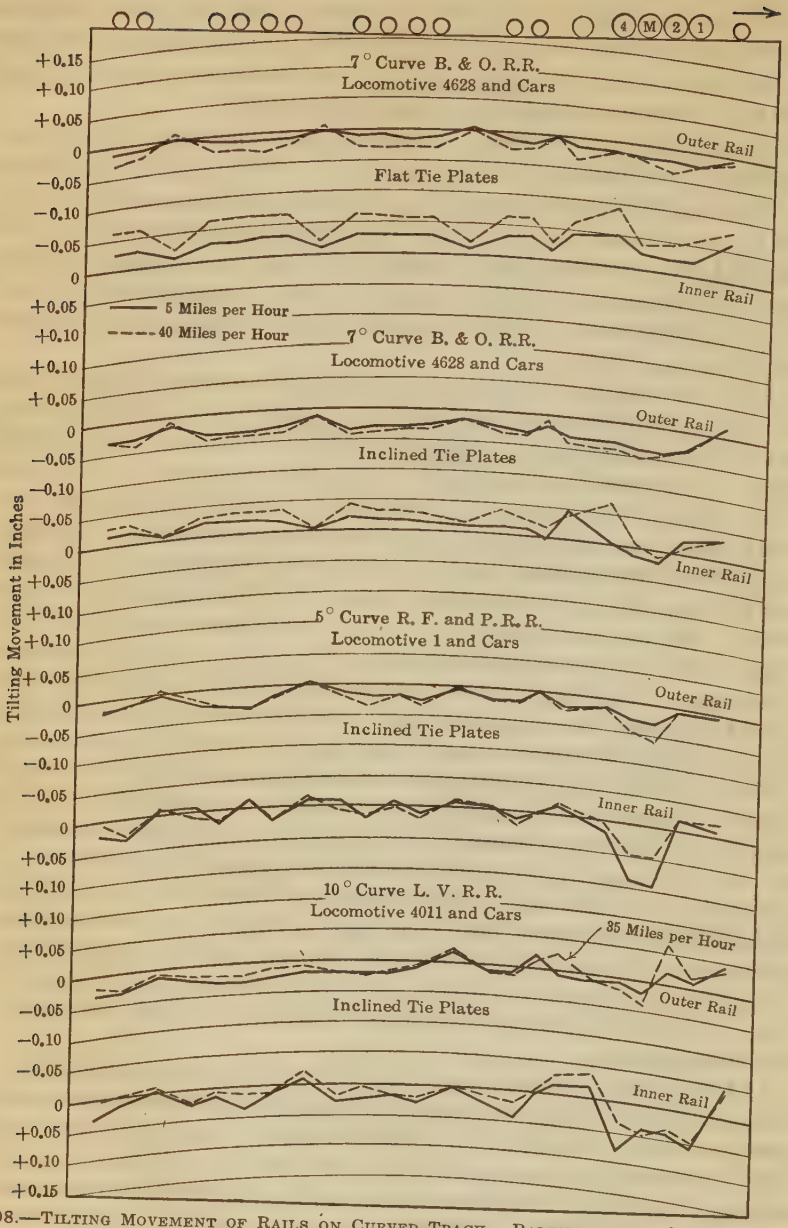


FIG. 98.—TILTING MOVEMENT OF RAILS ON CURVED TRACK. BALTIMORE AND OHIO RAILROAD, LEHIGH VALLEY RAILROAD, AND RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

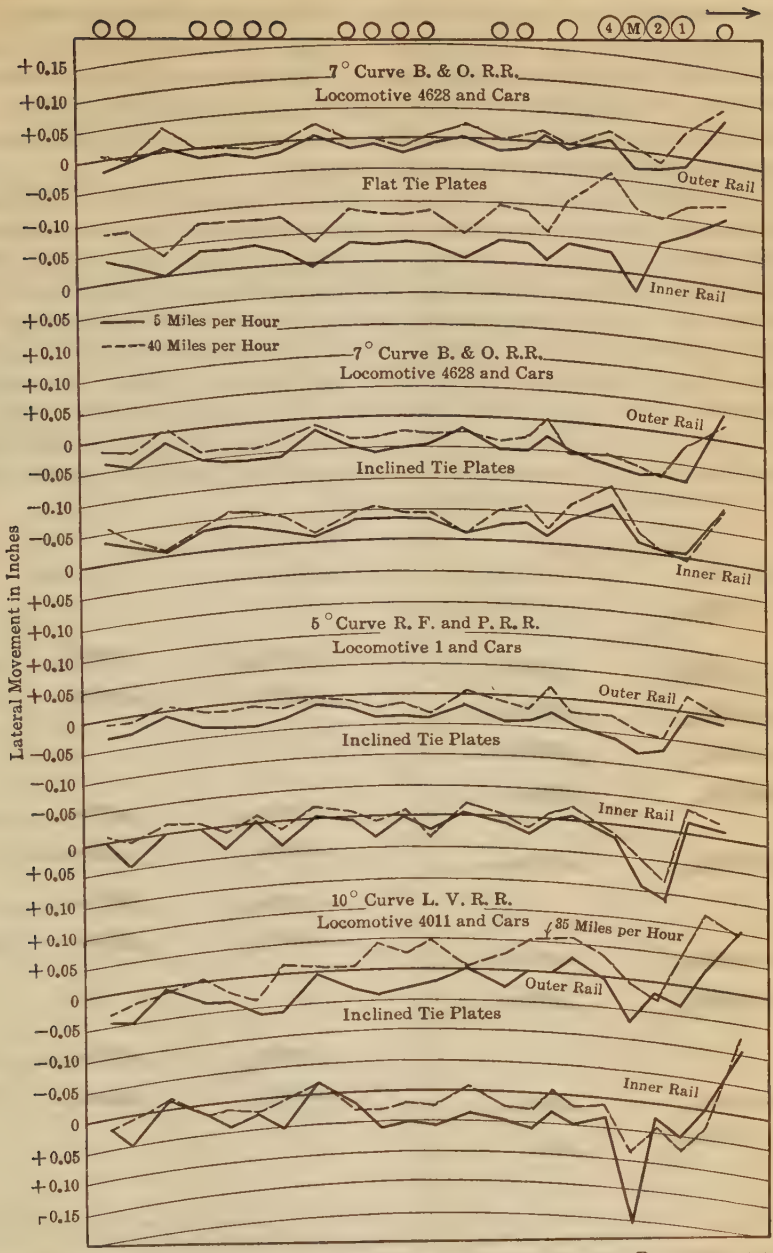


FIG. 99.—LATERAL MOVEMENT OF HEAD OF RAILS ON CURVED TRACK. BALTIMORE AND OHIO RAILROAD, LEHIGH VALLEY RAILROAD, AND RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

and fifth drivers. On the two curves of the Richmond, Fredericksburg and Potomac Railroad, the head of the inner rail moved outwardly 0.10 in., or more, under the first and second drivers of the Pacific type locomotive. It will be noted that the lateral movement of the rail on all these curves was considerably less than that found on the tests on the Atchison, Topeka and Santa Fé Railway and the Chicago, Milwaukee and St. Paul Railway, as may be expected from the difference in rail sections.

31.—Position of Bearing of Wheel on Rail on Curved Track.—The flattenings of copper wire that were taken on the curved track were measured and the diameter of the wire subtracted as already described for straight track. It is believed that the method used will give a fairly correct idea of the position of the center of the bearing of wheel on rail. The values found at the higher speeds are plotted in Figs. 100 to 102, and at 5 miles per hour in the case where impressions at the higher speed were not taken. The vertical lines represent the vertical axis of the rail. No impressions were taken on the 7° curve of the Baltimore and Ohio Railroad laid with inclined tie-plates; there was no opportunity to replenish the supply of copper wire within the time the test train was available.

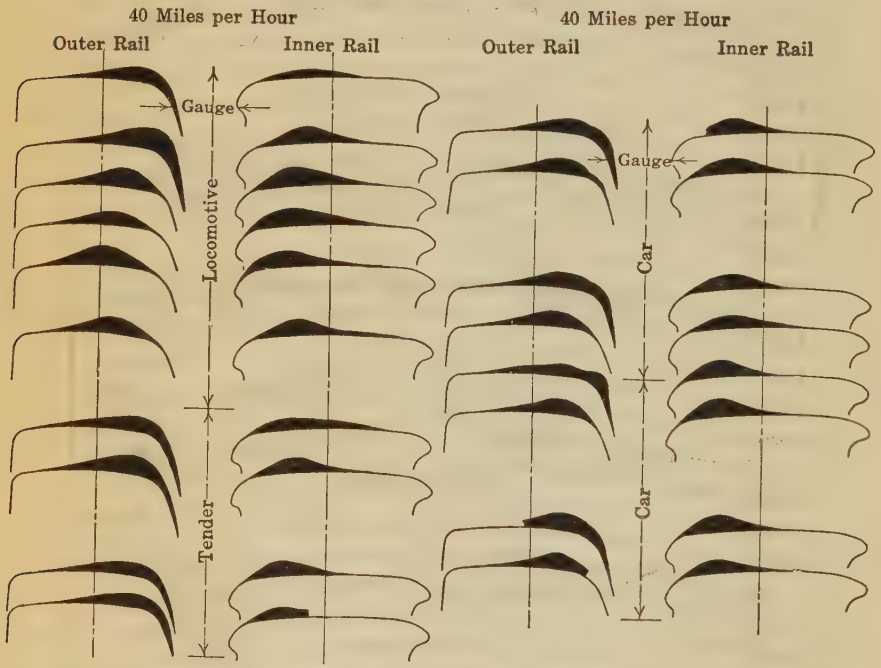


FIG. 100.—BEARING OF WHEEL ON RAIL OF 7° CURVE. FLAT TIE-PLATES. MIKADO TYPE LOCOMOTIVE AND CARS. BALTIMORE AND OHIO RAILROAD.

It may be expected that there will be some variation in the position of the wheel bearing on a given curve—especially that one wheel of a truck or frame will differ from another. The results, however, seem quite consistent. Describing the position of the center of bearing in terms of the width of the

flatter portion of the top of the head (this is not considering the curves at the corners) and counting always from the gauge side of the rail, a study of the diagrams will show that the center of pressure on the outer rail of the 7° curve of the Baltimore and Ohio Railroad laid with flat tie-plates for all the wheels of the locomotive and cars usually lies at from one-fourth to one-third the width of head, except that for the fourth driver 0.45 will denote the position, and on the inner rail one-fourth to one-third will also represent the relative position of the center of pressure, both rails having a slight outward cant. On the 10° curve of the Lehigh Valley Railroad, at a speed of 35 miles per hour, the position of the center of bearing on the outer rail (cant, 1 in 36) was at from one-third to one-half the width for all except the third driver (flangeless), which was at three-fourths the width. On the inner rail (cant, 1 in 27), the position was usually near four-tenths the width except for the third driver (flangeless), which was 0.55, counting in all cases the full width of an unworn rail. On the 2½° curve of the Richmond, Fredericksburg and Potomac Railroad, at 5 miles per hour, the position on the outer rail (cant slightly outward) was one-third to one-half, except for the third driver (worn tire), which was two-thirds. On the inner rail (outward cant, 1 in 32), the position was at one-fourth to one-third. On the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, the position on the outer rail (cant, 1 in 26) was close to one-half, except that it was three-fourths for the second and third drivers (worn tires). On the inner rail (cant, 1 in 42), the position was one-half or more, except for the second and third drivers with the worn tires resting on the outer part of the head. In general, there was little difference in the position of the bearing at the high and the low speed, even though at one speed the flange of the wheel might be closer to one rail than to the other. The position of the wheels on the outer rail generally showed flange pressure for the guiding wheels of the driver group and for the first wheel of the trucks of the tender and cars.

The position of the wear of the rail on the curved track is of interest. On the 7° curve of the Baltimore and Ohio Railroad laid with inclined tie-plates, both the outer and the inner rail showed brightness and wear across the full width of the head. As the 7° curve with flat tie-plates had been used with inclined tie-plates to within a few days of the test, the wear was that found with inclined tie-plates. Of the 1° curve of the Reading Company on which measurements of the movement of the rail were made, and which had been laid in the two ways two months before for the purposes of observation, the part laid with flat tie-plates showed brightness over two-thirds of the width of the head of the outer rail (inward cant, 1 in 60) and three-fifths of the inner rail (outward cant, 1 in 30), and the part laid with inclined tie-plates showed brightness over the full width of the outer rail (cant, 1 in 15) and five-sixths of the width of the inner rail (cant, 1 in 24), the appearence of the outer rail laid with inclined tie-plates indicating that the greatest bearing came slightly outside the middle of the head. On the 10° curve of the Lehigh Valley Railroad, laid with inclined tie-plates, the brightness was over nearly the full width of the outer rail (cant, 1 in 36) and the greatest possible width of the worn inner rail (cant, 1 in 27). The outside of the

FIG. 101.—BEARING OF WHEEL ON RAIL OF 10° CURVE. INCLINED TIE-PLATES. SANTA FÉ TYPE LOCOMOTIVE AND CARS. LEHIGH VALLEY RAILROAD.

FIG. 102.—BEARING OF WHEEL ON RAIL OF 2½° CURVE, FLAT TIE-PLATES, AND 5° CURVE, INCLINED TIE-PLATES. PACIFIC TYPE LOCOMOTIVE AND CARS. RICHMOND, FREDERICKSBURG AND POTOMAC RAILROAD.

head of the inner rail (re-laid from an outer rail) was so worn that the center of the whole bearing area of the head was necessarily to one side of the center line of the web. On the 5° curve of the Richmond, Fredericksburg and Potomac Railroad, laid with inclined tie-plates, the brightness extended over three-fourths of the width of the outer rail (cant, 1 in 26), the gauge side of the head being somewhat worn, and on the inner rail (cant, 1 in 42), the maximum wear was a little inside the middle of the head.

Estimates were made of the position of the center of wheel bearing for the various wheels of the locomotive, tender, and cars, and an average value of the distance of these centers from the middle of the rail head was found. The average position of the center of bearing shown by the impressions on copper wire was calculated, but some consideration was given to the position of the brightness of the rail, and when the wire impressions were not available the other observations were used. The values so estimated are given in Column (3) of Table 17; they are considered to be more accurate than the preceding general statements. In making up Table 17, the position of the wheels and drivers most actively engaged in forming the guiding action was not taken into consideration.

32.—Discussion of Canted Rail and of Unsymmetrical Tie-Plates on Curved Track.—The study of the data in the preceding articles has shown that the center of the bearing of the wheel on the rails of the curved track tested is more nearly at the middle of the head of the rail when the rail is canted inwardly than when it is vertical. It is apparent from the tests that a cant of 1 in 20 will give a satisfactory bearing position for the wheels of the locomotives and cars used in the tests. The position of the wheel bearing does not differ greatly whether or not the flange of the wheel runs close to the rail, except in the case of locomotives having worn driver tires. In the case of rail that is normal or nearly normal to the transverse section of the track, the position of the center of wheel bearing ranged from 0.25 to 0.60 in. inwardly of the middle of the head of the rail, 0.50 in. being a common value. It is also apparent that in curved track as in straight track a cant of 1 in 20 will bring the center of the wheel bearing close to the middle of the head of the rail. The appearance of brightness over the head of the rail and the distribution of the wear also favored the canted rail. This is well shown in the 1° curve of the Reading Company, laid for the purpose of observation on the effect of canting, where the part laid with inclined tie-plates showed brightness over the full width of the head and that laid with flat tie-plates showed brightness only over two-thirds and three-fifths of the width. It was also found that on the two 7° curves of the Baltimore and Ohio Railroad the lateral bending stresses in the rail laid with inclined tie-plates were somewhat less than those found when flat tie-plates were used. It is not apparent that canting the rail will have any marked effect on the rapidity of wear of the gauge side of the outer rail by the flanges of wheels which bear against it and guide a truck or group of wheels in traversing the curve, except as the decrease in lateral bending stresses may show a reduction in flange pressures. Taken altogether the tests indicate that for curved track the canting of the rail conduces to a wider distribution of the bearing pressure on the head of the

rail and a more favorable distribution of wear, and thus may be expected to improve the conditions for track maintenance.

It is not apparent that canting of the rail has any marked effect on the tilting of the rail under load, or on changes in gauge by traffic, or on tie cutting. It should also be noted that the variation in lateral bending stresses for different runs and different points along the track is present both in track having vertical rail and in track having canted rail and to approximately the same degree.

Except for certain wheels of the locomotive, the action of the wheels was to tilt both outer and inner rail inwardly of the track a small amount, thus narrowing the gauge at the wheels. In the same way for the same wheels there was a movement of both rails inwardly of the track, also tending to narrow the gauge under these wheels. This condition was found to exist with both vertical and canted rail. This inward tilting and lateral movement of the rail may be attributed to the position of the tie-plate. If it is generally characteristic of such track, the inference may be drawn that for resisting the lateral forces present when this tilting takes place the eccentricity of the tie-plate is too great. Other circumstances point to another conclusion, at least for the inner rail.

It will be noted that in general the cant found on the curved track was less than that which would be expected with the tie-plates used. The inclined tie-plates had an inclination of 1 in 20—generally the rail was found to be canted less than this. The rail laid with flat tie-plates was usually found to be canted outwardly. It is reasonable to assume that when the rail was laid its position approached that which would be expected for the tie-plates used. This being true, it would seem that curved track tends to lose its cant under traffic, or to gain an outward cant, and thus to widen the gauge slightly. This change is opposite to that found in straight track, where the tendency is toward an increase in cant. It is also contrary to the tilting action found on curved track under most of the wheels. It seems evident, however, that there is a tendency for the rails of curved track to lose their cant or to become canted outwardly.

The reduction in cant may be due in part to the large outward pressures exerted on the outer rail by the flanges of the guiding wheels of the locomotive and on the inner rail by the intermediate drivers that resist the pressures exerted by the outer wheels of the locomotive. That these forces are important is borne out by the outward tilting and outward movement of the two rails under these wheels.

Another method of attack may lie in examining the probable eccentricity of application of the forces that act on curved track in somewhat the same way as was done with straight track, considering the point of application of the wheel load, the center of the tie-plate, the cant of the rail, and its super-elevation, etc. It is evident from a study of the lateral bending stresses that in addition to the lateral forces developed by the guiding action of groups of wheels there are outward lateral forces acting on the rail similar in nature to those giving an outward bending in straight track; the resultant of these forces and the vertical loads may be expected to have an outward trend. Although the magnitude of these lateral forces can not be determined, it will be well to

ascertain whether the conditions compare with straight track and what the effect of speed on the conditions of superelevated track will be.

In Table 17 are given values of the position of the wheel bearing on the rail, eccentricity of tie-plate, eccentricity due to cant of rail, eccentricity due to superelevation, etc., for the several curves on which tests were made. Fig. 103 will help in understanding the terms used. At the speed of superelevation the resultant of the centrifugal force and the vertical load may be considered to be normal to the transverse section of the track—parallel to the axis of the rail for flat tie-plates ($A\ B$, Fig. 103) and inclined to it for inclined tie-plates ($A'\ B'$). At the low speeds the centrifugal force is negligible and the resultant of the vertical load and the centrifugal force may be taken to be vertical ($A\ C$ and $A'\ C'$). The nominal eccentricity then is the distance from the middle of the tie-plate to its intersection with the resultant used. This nominal resultant ($A\ D$ and $A'\ D'$) is assumed to be acting along a line joining the center of the wheel bearing and the mid-point of the tie-plate. If the rolling action of the wheel gives an outward lateral force of a magnitude such that the resultant of this and the other forces coincides with the so-called nominal resultant, the conditions would be called very favorable. If, however, under one of the speeds the resultant of the actual forces can not be expected to occupy the assumed position, it will be evident that tilting forces will be present to modify the cant of the rail. The slope of the nominal resultant may then be used as a criterion. Even for another speed, the resultant may not coincide in position with the assumed nominal resultant.

It will be seen that for the speed of superelevation the nominal eccentricity ($B\ D$ or $B'\ D'$, Fig. 103) and the slope of the nominal resultant (tangent of angle between $A\ B$ and $A\ D$ or $A'\ B'$ and $A'\ D'$), as may be expected, for both rails would not differ from those for straight track provided the position of the wheel bearing and the cant of the rail were the same. The general slope of the nominal resultant for both inner and outer rails ranges generally from 0.11 to 0.15. It would appear that at the speed of superelevation, except for the guiding action of groups of wheels, the conditions for curved track will be much the same as for straight track.

At a slow speed it is seen that for the outer rail the line of the vertical load intersects the bottom of the tie-plate well inside its middle and that the nominal eccentricity ($C\ D$ or $C'\ D'$) is great, the values in Table 17 ranging from 0.82 to 1.78 in. The slope of the nominal resultant is correspondingly large—from 0.13 to 0.24. It is not to be expected that the outward lateral force on the rail produced by what has been called the outward rolling action of the wheels will be sufficient to give such a slope. With the actual resultant falling inside the mid-point of the tie-plate, the tendency will be for the outer rail to tilt inwardly as the load passes, except as the guiding flanges may give an outward pressure.

For the inner rail at a slow speed, the line of the vertical load intersects the bottom of the tie-plate generally not far from its mid-point, the nominal eccentricity ($C\ D$ or $C'\ D'$) given in Table 17 ranging from — 0.07 to 0.69 in. The values of the slope of the nominal resultant are very small. For a slow speed, then, the action of the traffic may be expected to tilt the inner rail outwardly, resulting in a decreased cant. This change will also be influenced

TABLE 17.—Center of Wheel Bearing, Eccentricity of Tie-Plate, Cant of Rail, Nominal Eccentricity, and Slope of Nominal Resultant on Curved Track.

(The values are given in inches, the positive direction being inward from the middle of the head of the rail. The position of the center of wheel bearing was estimated from the impressions of copper wire for the wheels of locomotive and cars when available and otherwise from the appearance of the rail and other observations. The eccentricity due to superelevation is that distance along the tie which corresponds to the height of the rail and thickness of tie-plates for a flat tie-plate. The other terms are the same as have been used for straight track.)

Location.	Average cant of rail.	Center of wheel bearing.	Eccentricity of tie-plate.	Eccentricity due to cant of rail.	Eccentricity due to superelevation.	At Speed of Superelevation.		At Slow Speed.	
						Nominal eccentricity.	Slope of nominal resultant.	Nominal eccentricity.	Slope of nominal resultant.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Outer Rail.									
Baltimore & Ohio R. R.:									
7° Curve:									
Flat tie-plate	—1:120	0.50	0.50	—0.06	0.84	0.94	0.13	1.78	0.24
Inclined tie-plate	1:23	0.00	0.50	0.31	0.86	0.81	0.11	1.67	0.23
Reading Co.:									
1° Curve:									
Flat tie-plate	1:60	0.45	0.50	0.12	0.22	1.07	0.15	1.29	0.17
Inclined tie-plate	1:15	—0.10	0.50	0.48	0.21	0.88	0.12	1.09	0.15
Lehigh Valley R. R.:									
10° Curve:									
Inclined tie-plate	1:36	0.00	0.13	0.21	0.78	0.84	0.05	1.12	0.15
Richmond, Fredericksburg & Potomac R. R.:									
2⅓° Curve:									
Flat tie-plate	—1:115	0.20	0.25	—0.05	0.42	0.40	0.06	0.82	0.13
5° Curve:									
Inclined tie-plate	1:26	0.00	0.50	0.25	0.59	0.75	0.12	1.34	0.21

TABLE 17.—(*Continued*).

Location.	Average cant of rail.	Center of wheel bearing.	Eccentricity of tie-plate.	Eccentricity due to cant of rail.	Eccentricity due to super-elevation.	At Speed of Super-elevation.		At Slow Speed.	
						Nominal eccentricity.	Slope of nominal resultant.	Nominal eccentricity.	Slope of nominal resultant.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
				Inner Rail.					
Baltimore & Ohio R. R.:									
7° Curve:									
Flat tie-plate	—1 : 120	0.60	0.50	—0.06	—0.84	1.04	0.14	0.20	0.03
Inclined tie-plate	1 : 23	0.00	0.50	0.31	—0.86	0.81	0.11	—0.05	—0.01
Reading Co.:									
1° Curve;									
Flat tie-plate	—1 : 30	0.55	0.50	—0.25	—0.22	0.80	0.11	0.58	0.08
Inclined tie-plate	1 : 24	0.10	0.50	0.30	—0.21	0.90	0.12	0.69	0.10
Lehigh Valley R. R.:									
10° Curve:									
Inclined tie-plate	1 : 27	0.30	0.13	0.28	—0.78	0.71	0.09	—0.07	—0.01
Richmond, Fredericksburg & Potomac R. R.:									
2½° Curve:									
Flat tie-plate	—1 : 32	0.40	0.25	—0.20	—0.42	0.45	0.07	0.08	0.00
5° Curve:									
Inclined tie-plate	1 : 42	0.00	0.50	0.15	—0.59	0.65	0.10	0.06	0.01

by the strong outward pressure on the inner rail exerted by the intermediate drivers and by other wheels, as is brought out in the study of the tests. The passage of trains at slow speeds and the outward pressure of certain wheels at both low and high speeds may be taken to be the cause of the loss of cant on all the curves. It would appear that for the inner rail there would be an advantage in having a greater eccentricity of tie-plate if many trains are to be run at speeds below that corresponding to the superelevation. Although in the tests shown in Fig. 98, the inner rail generally tilted inwardly under load, it is seen that the amount of the inward tilt at the higher speed was greater than that at the lower; if then other conditions caused an outward tilt, the effect would be the greater at the low speed.

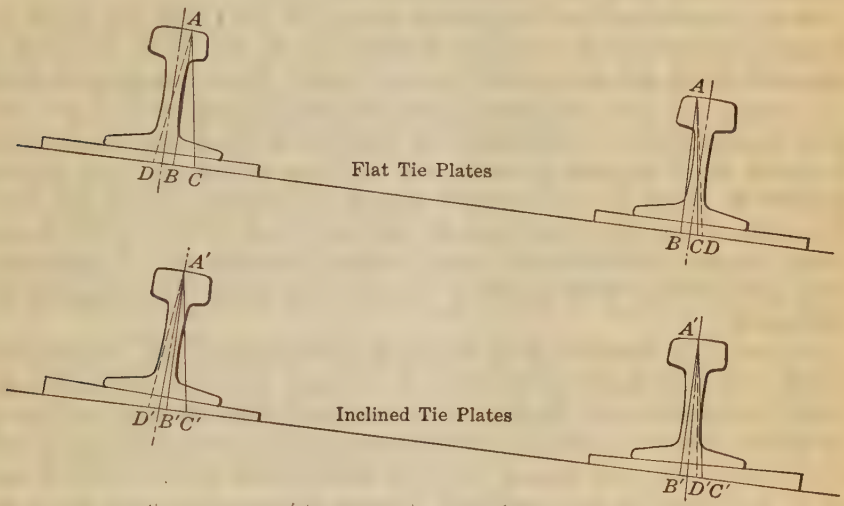


Fig. 103.—Nominal Eccentricity of Tie-Plates for Slow Speed and Speed Corresponding to Superelevation.

Any tendency to changes in cant that may be made at speeds higher than those corresponding to the superelevation will not need consideration, since the number of applications of load and their amount will be relatively small.

In the matter of the proper amount of eccentricity to be given in the design of tie-plates, it would seem that this can best be determined in practice by the results of experience on such track as that on which the tests were conducted. If it is found that the cant of the rail is decreased under service, it would be proper to increase the eccentricity of the tie-plate; if the cant shows a gain, the eccentricity may be too great. On the track used in the tests in a general way it may be said that the cant had increased on straight track and decreased on curved track.

As in the case of straight track it is of prime importance to have tie-plates of adequate length. Increasing the length will not only give increased bearing area, but the greater length will have a beneficial effect by reason of the decreased variation in pressure at the ends of the tie-plate corresponding to the variation in the direction of the resultant wheel load under the diverse conditions of traffic. In any case, the thickness of tie-plate must be sufficient for the purpose.

33.—Stresses in Heavy Rail.—The average values of the stresses in the 130-lb. rail under all the drivers of the locomotives were generally in accordance with values calculated by the analytical method given in the first progress report. Different track locations gave small differences in the average values, due partly to differences in the relative values of the positive and negative moments at and between wheels at the different locations. The values of the stresses under the individual drivers differed from the analytical results, showing a variation in the division of the load from the reported loads. As the stresses were relatively small, the variation from average values may be expected to be greater than for the lighter rail, due both to the variations in the track and to observational error.

It has been shown that the average stresses in the 130-lb. rail under the drivers on straight track were about 8 000 lb. per sq. in. These increased to 9 000 or 9 500 lb. per sq. in. at 40 miles per hour. The increase due to speed was variable, but small, the average increases ranging from 12 to 20 per cent. Mean stresses in base of rail under the drivers were found as great as 16 500 lb. per sq. in. at 40 miles per hour, and the stress at one edge was as great as 22 000 lb. per sq. in. The range of individual observations above and below the average mean stress was generally not more than 3 500 or 4 000 lb. per sq. in. At each instrument the range was much less than this, in some cases being not more than 1 000 lb. per sq. in. above and below the average for the instrument.

As would be expected from analytical considerations, the intermediate wheels of the two trucks at the ends of two cars, or of car and tender, developed lower stresses in the rail on straight track than the first and last of the group. On the Baltimore and Ohio Railroad, the mean stresses in base of rail under the wheel load of 18 000 lb. ranged from 5 000 to 7 500 lb. per sq. in. at 5 miles per hour and 5 500 to 9 000 lb. per sq. in. at 40 miles per hour. On the Reading Company the mean stresses under the wheel load of 25 000 lb. ranged from 6 000 to 10 000 lb. per sq. in. at 5 miles per hour and 6 000 to 11 000 lb. per sq. in. at 40 miles per hour. The effect of speed in the latter case was very low. It is seen that with the wheel load of 25 000 lb. the stresses in rail on straight track were nearly as great as any found under the drivers of the locomotive.

In the outer rail of the 7° curves of the Baltimore and Ohio Railroad average vertical bending stresses of 9 000 and 10 000 lb. per sq. in. were developed under a driver of the Mikado type locomotive at speeds of 5 and 40 miles per hour, respectively, and average stresses at one edge of 12 000 and 14 000 lb. per sq. in. at the two speeds. In the inner rail average vertical bending stresses of 13 500 and 11 000 lb. per sq. in. were developed at speeds of 5 and 40 miles per hour, respectively, and average stresses at one edge of 23 000 and 14 000 lb. per sq. in. at the two speeds. Comment may be made that except for the stress of 23 000 lb. per sq. in. the stresses on the 7° curve were relatively low. In the outer rail of the 10° curve of the Lehigh Valley Railroad the highest average vertical bending stress under a driver of the Santa Fé type locomotive was 11 200 lb. per sq. in. at 35 miles per hour. In the inner rail an average vertical bending stress of 21 500 lb. per sq. in. and

an average stress at one edge of 31 500 lb. per sq. in. were found at 5 miles per hour. At 35 miles per hour the corresponding stresses under this driver were 11 000 and 12 000 lb. per sq. in. It should be borne in mind that this inner rail was badly worn, having been relaid from the outer rail.

The cars with wheel loads of 18 000 lb. developed stresses in both rails of the 7° curves of the Baltimore and Ohio Railroad that were lower than the stresses found under the drivers, both vertical bending stresses and stresses at one edge. Similarly, on the 10° curve of the Lehigh Valley Railroad the wheel load of 24 000 lb. developed stresses that were considerably smaller than those found under the drivers of the Santa Fé type locomotive.

It is to be expected that the smaller lateral bending stress in the heavy rail, as well as the smaller vertical bending stress, will give a more general distribution of the lateral and vertical pressure on the ties and ballast, and thus act to decrease the maintenance work required.

34.—Other Tests.—Of the other investigations not reported here, further progress has been made in tests of rail joints. Measurements of strains in angle-bars under the load of wheels have been made on track. Laboratory tests on several of the forms of rail joint commonly used in track have been made from time to time as opportunity afforded, and thus data on the action of rail joints are being accumulated. Tests have been made to find the longitudinal pressure required to cause the rail to slip in the joint for different conditions, such as varied tension in the bolts. It is planned to carry further the investigation on rail joints. It is hoped that an instrument may be devised to measure strains in the splice-bars at the top, bottom, and middle of the bar under the conditions of support of rail found in track. Tests in other lines are also under consideration.

Respectfully submitted,

Special Committee to Report on Stresses in Railroad Track,

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December 23, 1924.

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Paper No. 1566

PROGRESS REPORT OF THE SPECIAL COMMITTEE ON HYDRAULICS PHENOMENA*

To the Board of Direction,
American Society of Civil Engineers:

Gentlemen.—The Committee has held one meeting during the year for the purpose of considering the plans for its work, and going over the results already obtained. It has also engaged in a voluminous correspondence.

The Committee feels that among the conditions that tend to hamper and limit the hydraulic research now being carried out by many different individuals in various locations scattered all over the United States, the following three are important:

First.—The lack of close contact among investigators with the resulting lack of that contagious enthusiasm and mutual interest which are so necessary to encourage and stimulate the worker to overcome difficulties and push a laborious undertaking forward to a successful conclusion.

Second.—A lack of experience and advisory direction with a consequent lack of advance appreciation of the difficulties that will have to be overcome in the course of an investigation.

Third.—Insufficient knowledge of work previously performed in the same field.

With the hope of lessening these obstacles the Committee has prepared a brief paper on the scope and distribution of current and recent research in hydraulics in the United States. (See Appendix.) This will enable any one contemplating or engaged on an investigation to know who else is working in the same field. Thus, workers may correspond with regard to the plans for their work and its progress, may avoid useless duplication, and may arrange for independent verification of results, a matter of great importance

* Presented to the Annual Meeting, January 21, 1925.

in most hydraulic work. Again, through this means, co-operative investigations may bring about more rapid progress.

The Committee has begun the preparation of a list of the names and locations of printed works on hydraulic subjects published before 1860 and now existing anywhere in this country. The completion and publication of this list will facilitate the consultation of these rare works by investigators.

The Committee has also undertaken to prepare evaluated digests of existing literature on a number of important topics, including: Water Hammer; Measurement of Water; Unsteady Flow in Open Channels; Flow in Pipes and Open Channels; Curved and Expanding Channels; and Recent Investigations of Nature of Fluids and Fluid Friction.

Although the work is divided among the different members of the Committee, it is of such nature as to require personal attention and much time. It will take some years to complete what has already been planned.

Respectfully submitted,

S. M. Woodward, *Chairman,*
M. L. Enger,
R. E. Horton,
A. T. Safford,
E. W. Schoder.

November 21, 1924.

APPENDIX

THE SCOPE AND DISTRIBUTION OF CURRENT AND RECENT RESEARCH IN HYDRAULICS IN THE UNITED STATES

Prepared by
The Special Committee on Hydraulics Phenomena
of the American Society of Civil Engineers, 1924.

The following list of research topics in hydraulics, accompanied by the names of active workers in each subject, has been prepared chiefly for two purposes:

First.—To enable different workers in similar or related lines to keep in touch with each other during the planning and progress of their researches.

Second.—To inform any engineer interested in some particular field where to apply to secure information regarding the most recent progress in that field.

Hydraulics touches on such a wide variety of topics that it has seemed best in this list to exclude in general the following fields: Hydrology, sanitary engineering, irrigation and drainage, and work with hydraulic machinery

having only a limited commercial interest. This exclusion, however, does not extend to any detail which is usually recognized as belonging to the general subject of hydraulics.

Experimental work carried on by students in educational institutions is not included, except in cases where the results were sufficiently reliable, complete, and satisfactory as to justify publication. For such cases the topic is listed over the name of the instructor having charge of the work, except where the work showed unusual initiative and independence on the part of the experimenter.

To keep the number of topics conveniently small and to bring closely related subjects together, the headings used are to be understood as rather general in scope.

The list has been obtained by correspondence lasting nearly a year and involving hundreds of letters. Although all the laboratories named in *Publication No. 5* of Engineering Foundation were addressed, as well as many names of individuals suggested from various sources, it can scarcely be hoped that the list is entirely complete. It is hoped that the number of omissions is small.

For those whose addresses are included in the 1925 Year Book of the Society, the names only are given; for others, whose addresses are given in the 1924 Year Book of the American Society of Mechanical Engineers, the names are followed by the letters, A. S. M. E. For those who are in neither of these classes, the names are followed by the full address.

Air-Lift Pump.—C. N. Ward, Madison, Wis.

Centrifugal Pumps.—R. L. Daugherty, A. S. M. E.; W. B. Gregory; L. F. Moody, A. S. M. E.; W. S. Pardoe; Allen F. Sherzer, Univ. of Michigan, Ann Arbor, Mich.; G. E. P. Smith; H. B. Taylor, A. S. M. E.

Current Meters.—C. M. Allen; R. L. Parshall.

Draft-Tubes.—C. M. Allen; R. L. Daugherty, A. S. M. E.; E. A. Dow; N. R. Gibson; L. F. Harza; L. F. Moody, A. S. M. E.; H. B. Taylor, A. S. M. E.

Effect of Temperature on Friction.—R. L. Daugherty, A. S. M. E.; Allen Hazen; F. E. Giesecke.

Flow in Improved and Artificial Channels.—H. K. Barrows; C. H. Eiffert; L. F. Harza; Allen Hazen; Julian Hinds; R. E. Horton; Thaddeus Merriman; Lynn Perry; C. E. Ramser; F. S. Scobey; Charles W. Sherman.

Flow in Natural Channels.—C. I. Corp, A. S. M. E.; N. R. Gibson; R. E. Horton; I. E. Houk; C. E. Ramser; F. C. Shenehon.

Flow Over Weirs.—C. M. Allen; H. K. Barrows; F. W. Greve, Purdue Univ., Lafayette, Ind.; H. P. Hammond; C. W. Harris, Univ. of Washington, Seattle, Wash.; L. F. Harza; Allen Hazen; Clemens Herschel; Julian Hinds; R. E. Horton; J. O. Jones; W. H. Kenerson, A. S. M. E.; Karl R. Kennison; Ford Kurtz; R. L. Parshall; F. A. Nagler; E. W. Schoder.

Flow Through Long Concrete Conduits.—C. H. Eiffert; John Ericson.

Flow Through Contractions in Open Channels.—L. F. Harza; Allen Hazen; E. W. Lane.

Flow Through Culverts.—F. A. Nagler; H. A. Thomas, Carnegie Inst. of Technology, Pittsburgh, Pa.; D. L. Yarnell.

Flow Through Head-Gates.—H. K. Barrows; Julian Hinds; R. E. Horton; R. L. Parshall.

Flow Through Orifices.—F. W. Greve, Purdue Univ., Lafayette, Ind.; Allen Hazen; Julian Hinds; J. O. Jones; H. H. Jordan; E. W. Lane; R. T. Livingston, A. S. M. E.; W. S. Pardoe; R. L. Parshall; Fred B. Seely, Urbana, Ill.

Flow Through Pipes.—H. K. Barrows; C. I. Corp, A. S. M. E.; W. F. Durand, A. S. M. E.; M. L. Enger; John R. Freeman; N. R. Gibson; F. E. Giesecke; W. B. Gregory; F. W. Greve, Purdue Univ., Lafayette, Ind.; Allen Hazen; Karl R. Kennison; Thaddeus Merriman; W. S. Pardoe; Lynn Perry; D. W. Proebstel, Portland Elec. Power Co., Portland, Ore.; C. E. Ramser; E. W. Schoder; F. S. Scobey; D. L. Yarnell.

Flow Through Pipe Fittings.—H. E. Babbitt; C. I. Corp, A. S. M. E.; E. A. Dow; M. L. Enger; John R. Freeman; F. E. Giesecke; F. W. Greve, Purdue Univ., Lafayette, Ind.; Allen Hazen; Lynn Perry; E. W. Schoder.

Hydraulic Jump.—John C. Beebe; Karl R. Kennison; F. A. Nagler; R. L. Parshall; R. M. Riegel; S. M. Woodward.

Hydraulic Ram.—L. F. Harza.

Loss of Head at Bridges.—F. A. Nagler.

Mixing of Streams Moving at Different Velocities.—William W. Tefft.

Non-Uniform and Unsteady Flow in Open Channels.—John R. Freeman; R. E. Horton; R. D. Johnson, A. S. M. E.; L. K. Sherman; H. A. Thomas, Carnegie Inst. of Technology, Pittsburgh, Pa.; S. M. Woodward.

Percolation Through and Under Earth Dams.—A. Lincoln Fellows, Denver, Colo.

Pitot Tubes.—C. M. Allen; W. S. Pardoe; D. W. Proebstel, Portland Elec. Power Co., Portland, Ore.; H. A. Thomas, Carnegie Inst. of Technology, Pittsburgh, Pa.

Pulsations in Piezometer Tubes.—F. A. Nagler; D. W. Proebstel, Portland Elec. Power Co., Portland, Ore.

Salt Velocity Method of Water Measurement.—C. M. Allen; H. K. Barrows.

Scouring Velocities.—C. A. Bock; John R. Freeman; Allen Hazen; Barton M. Jones, Pueblo Conservancy District, Pueblo, Colo.

Surge Tanks.—R. D. Johnson, A. S. M. E.; W. F. Durand, A. S. M. E.

Upward Pressure Under Dams.—E. L. Chandler; B. T. Colman; Julian Hinds; E. W. Lane.

Venturi Flume.—R. L. Parshall.

Venturi Meters.—C. M. Allen; Allen Hazen; W. H. Kenerson, A. S. M. E.; Clemens Herschel; W. S. Pardoe.

Water Turbines.—R. E. Horton; L. F. Moody, A. S. M. E.; H. B. Taylor, A. S. M. E.

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Paper No. 1567

HYDRO-ELECTRIC POWER AS A BY-PRODUCT OF AGRICULTURAL STORAGE*

By Everett N. Bryan,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. L. T. Guy, A. J. Wiley, and Everett N. Bryan.

Synopsis

The recent marked advance in the art of building high dams has made available a new source of power—a by-product of the release of water to satisfy an irrigation demand. These by-product power developments have characteristics peculiar to themselves, most important of which is the remarkable way in which the output curve follows the demand curve of adjacent irrigated areas requiring power for pumping. The ultimate amount of power that will be available from these plants will govern the agricultural development of areas which would be without water supply if it were not for this relatively cheap and abundant source of power for pumping. This paper further presents some of the unusual problems in the marketing of this power toward the solution of which little progress has yet been made, and some of the dangers connected with this type of development which should be recognized and guarded against.

A Comparatively New Type of Development

Engineering science has recently entered upon an era in which the operation of irrigation storage reservoirs is to deliver a considerable block of hydro-electric energy purely as a by-product. Although the United States Reclamation Service was the pioneer in this type of development, the first plant to be constructed in California was that recently completed at Don Pedro on the

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Tuolumne River by the Modesto and Turlock Irrigation Districts. Another similar plant is already in process of construction by the Merced Irrigation District at Exchequer on the Merced River. Plans are also now well under way for the construction of others at Melones on the Stanislaus River, at Millerton on the San Joaquin River, and at Pine Flat on the Kings River.

San Joaquin Valley is leading in this movement, as might be expected on account of its more advanced agricultural development and more urgent need of agricultural storage, but such a combined storage reservoir and power plant to be located at Iron Canyon near Red Bluff on the Sacramento River has been under consideration for a number of years. Although the present development of mining débris dams and power storage on the tributaries of the Sacramento River may retard the early development of strictly agricultural storage in the lower reaches of those tributaries, still the ultimate construction of such combination agricultural storage and power projects in the lower reaches of all the main tributaries of the Sacramento and San Joaquin Rivers is confidently expected.

There are certain peculiarities of these power plants and their output which place them in a class apart from all other installations. They are even distinctly different from such composite developments as the project of the Yuba River Power Company on the Yuba River, where irrigation, mining, and power are to benefit jointly, and in which case if the benefit to either enterprise may be considered a by-product, it is the benefit from stored water delivered for irrigation.

It is the purpose of this paper, first, to invite attention to certain peculiar characteristics of these power installations in which the output is purely a by-product of agricultural storage; second, to consider briefly the relative importance of these installations; third, to discuss certain conditions which surround the marketing of this by-product; and, fourth, to sound a warning concerning the dangers which beset this type of power development.

Characteristics of By-Product Power Plants

Effect of Recent Advance in Art of Building High Dams.—These by-product power developments have been made possible by recent rapid progress in the design and construction of high dams. The highest dam completed at the close of the Nineteenth Century was the Chartrain, or Tache, Dam, designed by French engineers and completed in 1892 with a height of 174.7 ft. Since that time leadership in the art has passed to American engineers, particularly the engineers of the U. S. Reclamation Service who have built several dams well over 200 ft. in height, one of which, the Arrowrock Dam, on Boise River in Idaho, has a total height of 348.6 ft. Now they are proposing something far more daring—a dam at Boulder Canyon on the Colorado River which would tower 735 ft., or more than four times the height of any that had been completed twenty-five years ago. It is this improvement in the design and construction of high dams which has made possible the huge agricultural storage reservoirs which will deliver hydro-electric power as a by-product.

Output Is Distinctly a By-Product.—These power installations are further peculiar in that the output is distinctly a by-product—purely incidental to the

operation of the storage works for the benefit of the lands to be irrigated. Such a power installation, being in a sense merely a stepchild, is controlled in its operation by considerations connected with the needs of the true child, agriculture, rather than by the exigencies of the power market. If the release of stored water for irrigation results in producing just sufficient power to satisfy the immediate demand for power it is well; if the release of stored water for the benefit of the land to be irrigated thereby will produce more power than is needed, the stored water must nevertheless be released—because the irrigation demand cannot wait; and if the release of stored water for the benefit of irrigation is insufficient to satisfy the instant demand for power then must the power demand go unsatisfied (if there is no standby plant) unless there is such an abundance of water that some may be used only for the generation of power and then be wasted.

Low Cost per Horse-Power Affects Output.—In the case of these by-product power installations the relatively small proportion of the total cost actually incident to the power development tends to exert a peculiar influence on the output. A statement of the Turlock Irrigation District (California), on January 1, 1922, proposes that $2 570 000 be expended on the Don Pedro Dam and only $1 028 000, or approximately $23 per h.p., on the power system. In the case of the Melones Project, on the Stanislaus River, the estimated total cost is $3 173 000, of which $1 516 000 represents the cost of the power installation, at a rate of approximately $47 per installed horse-power—a rather extreme condition because it is proposed to place the power plant not at the dam, as is usual, in order to avoid the high cost of long diversion conduits, but at a point about 1 mile down stream where an increase of 40 ft. in effective head can be obtained at what is thought to be a reasonable cost.

Ordinarily, the power plant is located at, or near, the storage dam, in which case all, or nearly all, the cost of producing the head on the turbines may be charged against the agricultural storage so that the unit cost of the power installation becomes very low. It makes little difference in the total cost of the project, therefore, whether the power installation is for a 40, a 50, or a 60% load factor. As might be expected this, in turn, has a marked influence on the possible output curve of the power plant.

Output Synchronizes with Irrigation Demand.—Naturally, the output curve of these by-product power installations—which as previously stated are the foster or stepchildren of the projects—takes on the peculiar characteristics of the demand curve of the true child, which is the irrigated area to be served with the stored water. How closely these two curves follow each other is indicated on Fig. 1 which is intended to show the percentage relations of the monthly distribution of irrigation demand (San Joaquin Valley), of the power output of by-product plants (San Joaquin Valley), and of the combined load of the largest three Californian utilities distributing electricity.

The curves of irrigation demand and power output are parallel and similar except that the peak of the irrigation demand is not reached until shortly after the peak of the power output has passed. They do synchronize to such an extent, however, that in cases of equal installed horse-power 77% of the gross

annual power output becomes available when and as needed for irrigation pumping. An additional 5% of the total power output of the by-product plant becomes available at a time when, under normal conditions, it will be most easily absorbed into the general power market to replace standby power generated by steam or otherwise. To the extent that the power output curve of the by-product plant falls below the irrigation demand curve in May, June, and July, such demand for peak power for irrigation pumping can most readily be taken care of by the ordinary hydro-electric plants supplying the general market, because such plants can then generate a maximum. For comparison only the monthly distribution of the combined load of the largest three utilities in California supplying electric power is shown on Fig. 1, this being a combined system load curve and hence supplemented by standby power.

Small Consideration Given to Auxiliary Service.—These by-product power projects are further peculiar in their lack of well formulated plans for auxiliary or standby service. They are seldom, if ever, considered as forming a part of the gigantic interconnected power system of the Pacific Coast by which electric energy—were it not for a few unimportant line gaps aggregating only about twenty-five miles—could be carried from the Canadian to the Mexican borders. Yet thus far but little thought has been given to the construction of auxiliary steam or other standby plants which could meet the demand at times of break-down or low water.

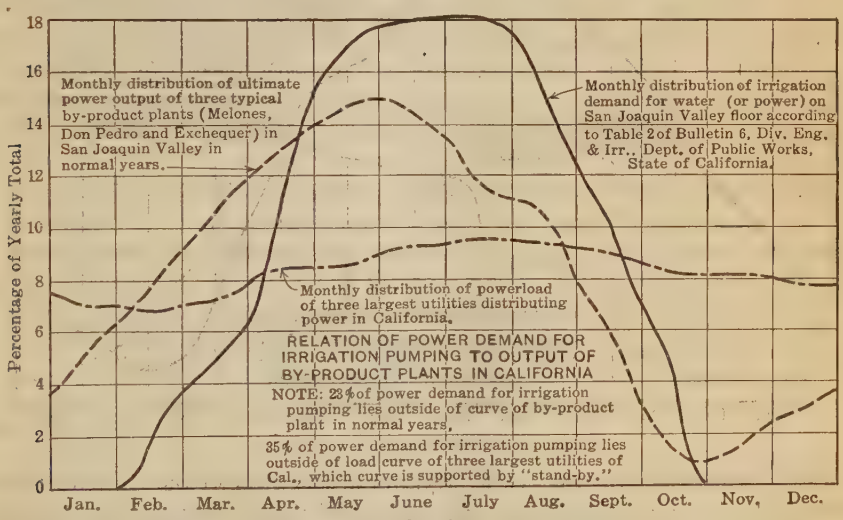
Percentage of Yearly Total
18
16
14
12
10
8
6
4
2
0
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Monthly distribution of ultimate power output of three typical by-product plants (Melones, Don Pedro and Exchequer) in San Joaquin Valley in normal years.
Monthly distribution of irrigation demand for water (or power) on San Joaquin Valley floor according to Table 2 of Bulletin 6, Div. Eng. & Irr., Dept. of Public Works, State of California.
Monthly distribution of powerload of three largest utilities distributing power in California.
RELATION OF POWER DEMAND FOR IRRIGATION PUMPING TO OUTPUT OF BY-PRODUCT PLANTS IN CALIFORNIA
NOTE: 23% of power demand for irrigation pumping lies outside of curve of by-product plant in normal years.
35% of power demand for irrigation pumping lies outside of load curve of three largest utilities of Cal., which curve is supported by "stand-by."

FIG. 1.

Lack of State Control.—This type of power development also enjoys a peculiar immunity from public regulation. In general, these developments will be made by irrigation districts which, in the matter of rates and service, are not subject to the State control exercised by the Railroad Commission over most distributors of electrical energy. As long as the power is distributed wholly within the boundaries of the producing district the wishes of the consumers will influence the officials of the power system, because the consumers

will also be the voters who can elect and recall the directing management; but when power is distributed outside the boundaries of the producing district there is no public authority, except the Courts, which can stand between producer and consumer.

Importance of This Type of Development

Installed Horse-Power Proposed.—The importance of this type of hydro-electric development, in which the energy is a by-product of agricultural storage, is often under-estimated. The installation recommended for the Melones Project on the Stanislaus River is 32 000 h.p. The Don Pedro Project on the Tuolumne River has three 5 000 kv-a. units installed, with provision in the plant and in the outlets constructed through the dam for the addition of four more similar units, giving a total capacity of about 45 000 h.p. At Exchequer, on the Merced River, an agricultural storage project is being constructed in connection with which it is proposed to build a power plant having a capacity of 32 000 h.p. Plans are in preparation for agricultural storage at Pine Flat on the Kings River, including a power plant of 65 000 h.p., or more, and numerous other similar projects are in contemplation in California.

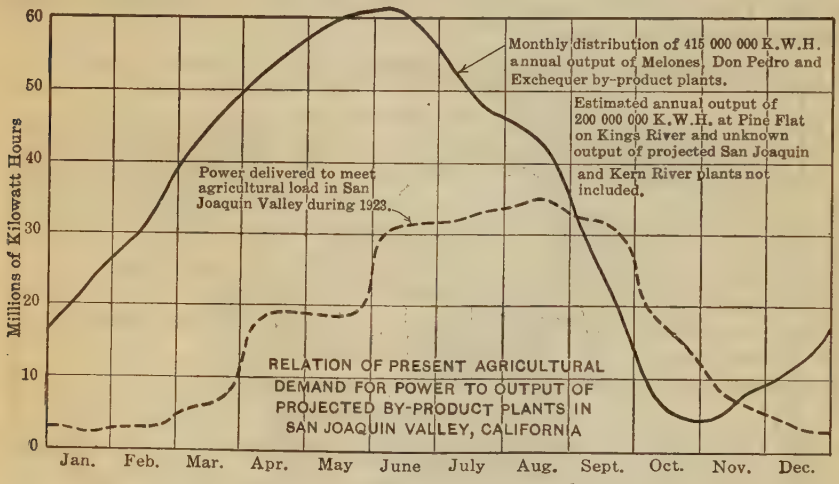
Millions of Kilowatt Hours
60
50
40
30
20
10
0
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Monthly distribution of 415 000 000 K.W.H. annual output of Melones, Don Pedro and Exchequer by-product plants.
Estimated annual output of 200 000 000 K.W.H. at Pine Flat on Kings River and unknown output of projected San Joaquin and Kern River plants not included.
Power delivered to meet agricultural load in San Joaquin Valley during 1923.
RELATION OF PRESENT AGRICULTURAL DEMAND FOR POWER TO OUTPUT OF PROJECTED BY-PRODUCT PLANTS IN SAN JOAQUIN VALLEY, CALIFORNIA

Fig. 2.

E. C. La Rue, M. Am. Soc. C. E., Hydraulic Engineer of the United States Geological Survey, is authority for the statement* that an installation of about 1 000 000 h.p. is practical at the proposed Lees Ferry Dam on the Colorado River, and that 4 000 000 continuous h.p. can be developed in the 670-mile stretch of the Colorado River between the Town of Green River, Utah, and Boulder Canyon, a short distance above the California line. This Colorado River development will probably be of a composite character, however, intended for flood protection, power, and irrigation, so that the project may not come within the narrow classification of this paper.

* *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 202.

Estimated Annual Output of Proposed Plants.—For normal years the annual output of the four plants in the San Joaquin Valley is estimated as follows: Melones, 109 000 000 kw-hr.; Don Pedro, 185 000 000 kw-hr.; Exchequer, 120 000 000 kw-hr.; and Pine Flat 200 000 000 kw-hr. These are only four of six similar plants proposed for the San Joaquin Valley, but their combined output as estimated is 614 000 000 kw-hr., which is almost three times the electrical energy consumed for agricultural purposes in the whole of the San Joaquin Valley from Bakersfield to Stockton during 1923. A comparison of the estimated combined output of the Melones, Don Pedro, and Exchequer plants in normal years, with the consumption of power for agricultural purposes in San Joaquin Valley in 1923, is shown in Fig. 2.

Relation of Installed Capacity to That of Irrigation Pumping Plants.—It is interesting to compare the proposed ultimate installed horse-power of these by-product plants with the installed horse-power of pumping plants elevating water to adjacent irrigated areas. The Melones, Don Pedro, and Exchequer Projects are intended to supply water by gravity to a gross area of about 587 197 acres and the total capacity of the three projected by-product power installations is 109 000 h.p., or roughly 0.18 h.p. per acre. On the opposite side of the San Joaquin Valley, within a range of 50 to 75 miles, there is an area of about 68 500 acres served by the Patterson Irrigated Farms Company, the Westside Irrigation District, the Byron-Bethany Irrigation District, and the East Contra Costa Irrigation Company which proposes an ultimate power installation of 8 800 h.p. (0.13 h.p. per acre) to pump water to a maximum height of 139 ft. The average lift to this latter area is unknown, but in an adjacent area of approximately 30 000 acres for which plans have recently been carefully drawn, 0.002 h.p. per acre per foot of lift will deliver 1 sec-ft. to each 110 acres.

The projected by-product power installation at Pine Flat, on Kings River, is comparatively much smaller than those at Melones, Don Pedro, and Exchequer, because the average yield of the Kings River is only about 2 acre-ft. per annum for each acre dependent on the river for water as against an average yield of 7.5 acre-ft. per annum by the Stanislaus, Tuolumne, and Merced Rivers on which the Melones, Don Pedro, and Exchequer plants are located. There is, therefore, no surplus water available for the generation of power alone and the proposed by-product power installation is only approximately 0.064 h.p. per acre.

These figures show that in the Northern San Joaquin Valley the areas now irrigated by gravity and already constructing, or about to construct, agricultural storage propose, in connection with their storage works, an ultimate by-product power installation equivalent in size to that required to satisfy the power demand of an equal irrigated area which must lift its water through an average height of 90 ft. When this is considered in connection with the further fact (see Fig. 1), that the output of these by-product power installations synchronizes to a remarkable degree with the demand for power for irrigation pumping, it is at once apparent how important this type of development is destined to become.

Disposition of By-Product Power

The marketing of this by-product power presents a difficult problem as yet unsolved. It is seasonal power (Figs. 1 and 2) of unequal monthly distribution. Either standby service must be arranged or the power must be delivered to a market the load variation of which synchronizes peculiarly with it. The widest difference of opinion prevails as to its probable value, because it is strictly a by-product and there is no certain way of determining its cost.

Wholesale Marketing.—The simplest course to pursue in marketing such power would be to sell the entire output of the plant, as has already been done by the Merced Irrigation District in the case of the power to be produced at Exchequer, thus passing on to the distributor the necessity of providing standby service. In such case, the prices received per kilowatt-hour will be relatively low.

Retail Distribution by Producer.—By a second alternative the producer of such power would undertake himself to distribute it. This is, in part, the course adopted by the Modesto and Turlock Irrigation Districts (Don Pedro plant) which have entered actively into competition within their own boundaries with the local public utility. They began without any standby plant or interconnection which would insure comsumers against interruption to service, but the Modesto Irrigation District has since undertaken the construction of a 1 250-kw. steam auxiliary plant. Until such provisions for emergencies are arranged, it is improbable that the installed capacity at Don Pedro will be greatly increased. The price of power should be relatively higher if the producer is to assume the responsibility of continuous service. The objection to this method is that it must necessarily draw the producer into the power business to an ever increasing degree and in unequal competition with large utilities having elaborate standbys and interconnections supplying a diversified load condition which it is difficult for the owner of the by-product plant to duplicate.

Expanding Project Area to Include Power Demand for Irrigation Pumping.—There is considerable promise in a third alternative as to the disposition of this by-product power. An assured market might be obtained for a considerable block of the power by constructing and operating the project either as a "Conservancy District" or as a "Public Utility District", which would include within the project boundary not only the area to be served by gravity with the stored water, but also an area requiring a considerable block of power for irrigation pumping. This alternative has already been suggested for the Melones Project, where the original proponents have little use for power and desire the construction of the dam for storage purposes. The taxpayers within the area to be served by gravity have been unwilling to undertake the financing of such an expensive project alone. The method by which it is proposed to distribute the cost and, at the same time, to gain an assured market for a large block of the by-product power is illustrative of what may be done on similar projects.

Melones Project as a Typical Case*

Original Project Defeated.—The two irrigation districts (Oakdale and South San Joaquin), which it was first proposed should build the Melones Storage Dam and Power Project, have a combined area of 145 196 acres. The net capacity of the storage reservoir is 103 500 acre-ft., and the estimated output of the power plant in normal years is approximately 109 000 000 kw-hr. The cost of the dam was estimated at $1 657 000 (exclusive of rights of way and preliminary investigations) and that of the power plant, $1 516 000. In order to build the dam and power-house, it would have been necessary, therefore, for the area of 145 196 acres within the original project boundary to have assumed a bonded indebtedness of $3 173 000, or $21.85 per acre, plus an amount for preliminary expenses, rights of way, and the power distribution system. In the original plan it was estimated that with the power plant operating on a load factor of 80% anything received in normal years over 1.98 mills per kw-hr. would produce a net income tending to reduce the cost of the stored water (which it was the real purpose of the project to obtain). However, a special tax levy for the purpose of purchasing the lands to be flooded by the impounded water was voted down by the two irrigation districts embracing the area within the original project boundary, presumably because it was felt the tax burden would be too great.

Enlarged Project Suggested.—As a result an enlarged project was suggested which would include within its boundary an additional 70 000 acres requiring power for pumping, with the maximum distance of transmission less than 70 miles. It was estimated that this enlarged area would require an ultimate installation of about 10 000 h.p. for irrigation pumping and have an annual consumption of approximately 30 000 000 kw-hr., and that in all probability within the time required for the construction of the Melones Project the annual demand within the enlarged area would have grown to at least 30 000 acre-ft. of stored water and 10 000 000 kw-hr. of power.

Disposal of Output and Income.—The present rates within this area are $0.75 per acre-ft. for stored water and a little less than $0.01 per kw-hr. for power for irrigation pumping. With due allowance for the present downward trend in this rate for power it is probable the annual demand of the project for stored water and power will cost more than $100 000 per year before the Melones storage and power works could be completed. At that time, there would be available in normal years a surplus of approximately 70 000 acre-ft. of stored water. This water could be used to regulate the flow of the river for the production of power during the season of greatest power shortage and this power, varying between 57 000 000 kw-hr. in dry years and 99 000 000 kw-hr. in normal years, could be sold to meet the difference between the $100 000 value of the benefit received (from the use of stored water and power within the project area) and the gross annual cost of the project, estimated at approximately $305 000.

* It is reported that a contract is about to be signed providing for the construction of this project by joint participation of the Pacific Gas and Electric Company and the two Irrigation Districts.

If the stored water and the power ultimately needed within the project area and satisfied by the project works are assumed to have the values of $0.60 per acre-ft. and 8 mills per kw-hr., respectively, the annual cost of the project will be met and there will remain for sale to other purchasers a block of power varying between 37 000 000 kw-hr. in dry years and 79 000 000 kw-hr. in normal years.

Power Apportionment.—An accompanying diagram (Fig. 3) shows the way in which the power load for irrigation pumping within the project area could ultimately be satisfied by the proposed works. Output curves of the power plant are shown both for normal and for dry years. It will be noted that not only is the demand of the included area for power for irrigation pumping satisfied in a substantial way without standby, but that the condition of the remainder of the output as regards the sale of the surplus to other parties is improved rather than injured by the demands of the project itself.

Value of By-Product Power Uncertain

The market value of this by-product power is as yet undetermined and is the subject of a wide difference of opinion. The market value may not easily be determined from the cost of production, because this cost is itself difficult, if not impossible, to determine. The power is a by-product and it is uncertain what proportion of the total cost of the project should be charged to agricultural storage and what proportion to power. Neither is there sufficient precedent to indicate the probable market value of such power, only two or three sales having been negotiated to date.

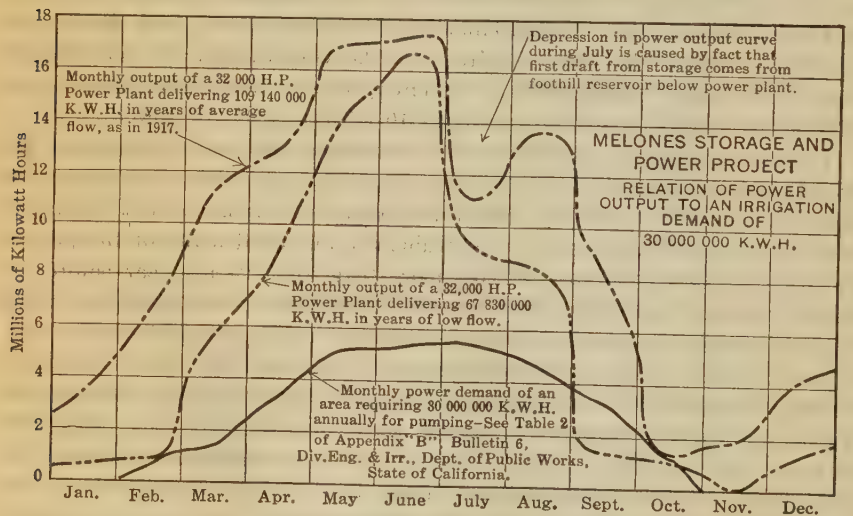
Millions of Kilowatt Hours
18
16
14
12
10
8
6
4
2
0
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Monthly output of a 32 000 H.P. Power Plant delivering 109 140 000 K.W.H. in years of average flow, as in 1917.
Depression in power output curve during July is caused by fact that first draft from storage comes from foothill reservoir below power plant.
MELONES STORAGE AND POWER PROJECT
RELATION OF POWER OUTPUT TO AN IRRIGATION DEMAND OF 30 000 000 K.W.H.
Monthly output of a 32,000 H.P. Power Plant delivering 67 830 000 K.W.H. in years of low flow.
Monthly power demand of an area requiring 30 000 000 K.W.H. annually for pumping—See Table 2 of Appendix "B", Bulletin 6, Div. Eng. & Irr., Dept. of Public Works, State of California.

Fig. 3.

Sale of Exchequer Plant Output.—The first sale of such power in California was negotiated by the Merced Irrigation District in 1921. The con-

tract provided that the District would install a generating capacity of 25 000 kw. at Exchequer and that the power company would pay for the output at the rate of 4½ mills delivered at the generator switch-board. This rate was fixed after an investigation by the Railroad Commission and received the approval of the Commission, of the purchasing company, and of the District. However, this rate should not be accepted with too much assurance as indicating the reasonable market value of such by-product power in other cases: It was the first block of such power offered for sale; it was to be generated within the territory of, and was purchased by, a utility which has a proportionately larger demand for power for agricultural pumping than any other utility in California; the contract offered the purchasing company a ready means of meeting an increasing demand for power without the expenditure of additional capital; and last, but probably by no means least, the contract offered to the purchasing company a means of intercepting the entry of a dangerous competitor into its territory.

Original Offer for Don Pedro Output.—On May 2, 1922, the Pacific Gas and Electric Company made an offer to the Modesto and Turlock Irrigation Districts for the Don Pedro power. This offer included several alternative rates, as follows:

(1) With an installed capacity of 15 000 kw., 4 mills for the first 7 000 000 kw-hr. per month and 3 mills for the remainder.
(2) With an installed capacity of 25 000 kw., 3 mills for the first 12 500 000 kw-hr. per month and 2.5 mills for the remainder.
(3) With an installed capacity of 35 000 kw., 2.5 mills for the first 18 000 000 kw-hr. per month and 2 mills for the remainder.
(4) If none of these proposals or modifications is agreeable, both parties to agree to accept a rate to be fixed by the Railroad Commission of the State of California.

None of these alternative proposals was acceptable to the irrigation districts and the negotiations were fruitless. The proposals are interesting, however, and indicate that generally speaking the larger the installed generating capacity for a particular project the more accentuated is the peak in the output curve and the lower becomes the average value per kilowatt-hour of output.

Partial Sale of Don Pedro Output.—Subsequently (March 11, 1924), the Turlock Irrigation District entered into a contract with the San Joaquin Light and Power Corporation for the disposal of a part of the power output of the Don Pedro Project. Interesting and important features of this contract are:

(1) The rate is 4.5 mills, but this applies to the output of only 6 500 kw. capacity out of the first 15 000 installed. Approximately 22% of the estimated annual output of the ultimate installation is involved.
(2) The output of any additional units which may be installed by the District (except as needed for consumption within the District boundaries) shall be sold to the power company at a rate to be fixed by the Railroad Commission of California.

(3) The District may deliver a maximum of 2 500 kw. during the period from January 1 to May 31 of each year and shall deliver a minimum of 6 500 kw. during the period from June 1 to December 31, except that in years of sub-normal stream flow the District is required to deliver only 65% of the output of its facilities during the period from August 1 to December 31.

(4) Delivery of power is to be made by the District at a point about 45 miles from the generating units.

(5) The agreement runs for 15 years and is renewable at the option of the District for another 15 years.

Sale of Nevada Irrigation District Power.—On April 29, 1924, the Nevada Irrigation District signed an agreement with the Pacific Gas and Electric Company arranging for joint use of certain canals owned by the Company. The Company is to enlarge its canals as necessary to carry the water of the District and will have the use of District water as it passes through power-houses en route. These power-houses are to be built by, or if already built, will be enlarged by, the Power Company which will own and operate them at its own expense. The water supply will be developed by the District which is to deliver a regulated flow of 220 cu. ft. per sec. during the 9 months beginning July 1 and ending March 31. For the power which thus can be developed, and for any power which can be developed as a substitute for steam power by the use of the District's water during the remainder of the year, the Company will pay during the first 25 years of the contract in varying ways and amounts and during the next 25 years at a uniform rate of 0.75 mill per kw-hr.

General Considerations Affecting Value.—In general, it is believed that those proposing the sale of this by-product power have approached the problem without a proper realization of the wide difference in value between potential station output at the generating switch-board and energy actually delivered to the consumer. The Railroad Commission of California made itself clear on this point when called upon to estimate the rate for the Exchequer plant.* It analyzed the data which had just been submitted in a rate case of the Southern California Edison Company, and found that hydro-electric power which cost 4.25 mills per kw-hr. at the generator switch-board, was worth 6.28 mills if the cost of standby service was included; 9.93 mills delivered to wholesale consumers at sub-stations if transmission losses and costs and general expenses were included; 18.9 mills delivered to actual consumers with installations of about 20 h.p.; and 74.0 mills delivered to residence consumers using approximately 20 kw-hr. per month for lighting.

Therefore, in any comparison of prices for this by-product power there should be borne in mind the conditions of the sale as regards (1) the use of all or a part only of the possible output; (2) the cost of providing standby service; (3) the cost of transmission to the point of delivery; and (4) the cost of collection of accounts.

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* See p. 918.

Perils of Proposed By-Product Developments

There are certain dangers of this type of development which should be recognized and guarded against.

False Encouragement of Irrigation.—Those familiar with the history of recent irrigation development in California need no reminder of the false promise which has been held out that water could be delivered great distances from those localities where there is an abundance of water to those localities where there is an inadequate supply merely by the sale of by-product power; nor of other premature agricultural development which has been encouraged by the hope that the sale of by-product power would pay for the cost of delivering an irrigation supply. The extent to which irrigation development should be subsidized by the income from by-product power deserves grave consideration.

Interference with Subsequent Development.—In general, these combined agricultural storage and by-product power developments are proposed and will doubtless be constructed, on the lower courses of the rivers. Unless some safeguard is provided, the priorities of right to use the water which are to be enjoyed by these developments may at a future date stand in the way of full development of the upper river. Any water loosed through these plants in excess of the instant irrigation demand down stream is forever lost to other irrigation and power use. The Division of Water Rights of the Department of Public Works, State of California, came to an early realization of this danger. The matter was discussed at the time of approving the applications to appropriate water for the Exchequer Power Project. Subsequently, when the Division was called to act on the applications to appropriate water for power purposes in connection with Don Pedro Project the District officials themselves acknowledged the danger and consented to a limitation in the permit, as follows:

"It is expressly recognized by all parties hereto that no rights to water, except stored water, are to be acquired under this permit which may in the future operate to the disadvantage of irrigation development."

Competition with Established Companies.—It is apparent that danger exists in the invasion of territory already served from existing power distribution systems by a competitor not subject to control in the matter of rates and service by some outside authority such as the Railroad Commission. For example, Don Pedro power is already being distributed within the project area in active competition with the utility serving that territory. This means duplication of distribution systems and consequent waste of capital. It means competition in which one party has already provided a full measure of insurance for its consumers against interrupted service and the other has not; in which one party is bound by a system of regular accounting not to subsidize one type of development at the expense of another, whereas the other is free and, in fact, enjoys a power of taxation with which to recoup losses. The danger is magnified when, and if, the producer of by-product power undertakes distribution outside the project area, because then there is no authority other than the Courts to intervene between the producer and consumer.

Summary and Conclusions

To recapitulate, the following facts have been demonstrated:

1.—A new era of development is dawning, one in which large blocks of hydro-electric power will become available as a by-product in connection with the operation of irrigation storage works.

2.—This by-product has certain peculiar characteristics in that (*a*) it is the result of an advance during the past twenty-five years in the art of constructing high dams; (*b*) it is purely a by-product—not an end in itself, but merely an incident to the development of agricultural storage; (*c*) it has a seasonal distribution strikingly akin to the seasonal distribution of the demand for agricultural power; (*d*) to date, there has been little progress in the direction of providing standby service; and (*e*) the producer of this energy enjoys generally an immunity from regulation in the matter of rates and service.

3.—The ultimate total delivery of power by these by-product plants is relatively very important. This power supply is adapted especially to the satisfaction of a demand for irrigation pumping. In the case of three large projects with an abundance of water, the proposed ultimate installation is equivalent to that required for an irrigated area of equal size with an average lift of 90 ft. This by-product would appear to open the way for irrigation of lands not heretofore thought susceptible of irrigation because of the cost of power.

4.—The marketing of this by-product presents some unusual and as yet unsolved problems. At least three means readily present themselves: (*a*) The power might be sold wholesale at the generator switch-board to a utility, which must bear the cost of standby service, of distribution, and of collection of accounts, and, therefore, can afford to pay only a relatively small part of the average rate paid by the actual consumers; (*b*) the producer may undertake himself to bear the cost of standby service, distribution, and collection of small accounts and may then expect a much higher average rate; and (*c*) the project area may be increased so as to include a demand for a considerable block of the by-product power for irrigation pumping, thereby distributing the capital cost of the project over a larger area, and leaving for outside sale a much smaller block of surplus power. Neither the cost of producing nor precedent establishes the sale value of such power; and the conditions which govern are not generally understood.

5.—Certain dangers connected with this type of development should be recognized and guarded against, such as false encouragement of premature and ill-considered irrigation development; the waste of water through low-head power plants along the lower reaches of the river, which water may be needed for irrigation and power development on the upper river; the destruction of capital by duplication of existing systems; and the lack of some disinterested regulative authority to stand between producer and consumer in the matter of rates and service.

DISCUSSION

L. T. Guy,* Jun. Am. Soc. C. E. (by letter).—This interesting outline of the problems usually encountered in hydro-electric developments in the United States, has led the writer to believe that a short account of a similar project in Australia may be of interest to members. This project is situated in the State of Victoria, Australia, and is known as the Sugarloaf-Rubicon Scheme, its construction having been authorized by Parliament.

In Victoria, irrigation and electric supply are controlled by independent Government authorities. The Sugarloaf Dam, which is approaching completion, is being constructed by the Irrigation Authority, and is of the rock-fill type with concrete core-wall. The river bed at the dam site is about Elevation 703, and the present construction comprises the impounding of water to Elevation 823, with a storage capacity of 300 000 acre-ft., provision also being made for an increase in height of full water level to Elevation 875 at some future time. It is proposed to locate the power station immediately below the dam.

The construction of the Sugarloaf Dam has been undertaken primarily for irrigation, and it is not proposed to charge any part of its cost to the electric supply undertaking. It is thus clear that the Electric Supply Authorities have no control over the discharge from the storage. The water discharged from the dam is not passed immediately to irrigation, but flows down the river for a distance of about 200 miles to a further regulating basin whence it flows by gravity to the land to be irrigated. There is thus no irrigation pumping load.

The irrigation season in Victoria extends in general from December to the following April, and water may possibly be passed during May. During this period, with the present storage capacity, it is proposed to maintain the irrigation flow at 833 sec-ft. continuously; however, the great distance between the dam and the actual offtake for irrigation is important, in that it will allow of the flow of water being varied throughout the day to follow the power load somewhat, provided the average daily flow is 833 sec-ft. After May, the outlets of the reservoir will be closed to allow the reservoir to fill in readiness for the following irrigation season, and the outlets will remain closed until full supply level is reached, which will normally occur about July or August. From then until the commencement of the next irrigation season, the reservoir is spilling continuously and the whole of such flow may be utilized for the generation of power.

It is thus obvious that for about three months in each year no power would be available from a power station situated at this dam, and although this station is interconnected with steam power stations of larger capacity it would not be feasible to develop such a plan, unless some economical means could be devised to bridge over this gap each year during the winter months of May, June, July, and sometimes August, or even later.

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Investigation disclosed the existence, within a few miles of the Sugarloaf Dam, of considerably smaller mountain streams, which could be developed as high-head plants and which reached their maximum flows during the months when power was not available from the Sugarloaf Dam. Conversely, the minimum flow of these streams occurs when power is available from the Sugarloaf Power Station, and although this minimum flow is considerably less than the proposed capacity of these plants, sufficient storage is obtainable at the intakes to serve as daily pondage. It will thus be possible to regulate the minimum daily flow so as to obtain the full output of the plant during the hours of peak loading. The total installed plant proposed on these mountain streams is 22 000 b.h.p. The auxiliary power, although relatively cheap, is naturally more expensive than that developed at the Sugarloaf Dam, but the combination enables a continuous supply of electric energy to be maintained at a favorable rate.

Gaugings are available at the Sugarloaf Dam dating from 1882—an exceptionally long period for Australia—and this has enabled careful consideration to be given to the size of plant to be installed, by the preparation of close estimates, both as to cost and power.

The exact size of the installation is not yet determined, but it will probably be about 12 000 b.h.p. At first sight, this capacity may sound excessive when it is considered that the regulated flow of 833 sec-ft. under the maximum head will produce only about 8 500 b.h.p. This increase is justified, however, by the large volume of water which is spilled from the reservoir during the months of August, September, October, and November, and is all available for power generation. This enables additional kilowatt-hours to be generated at a total cost less than the actual cost of coal required for the generation of similar power at a steam station.

Another problem encountered in a plant of this nature is the great range of head under which the turbines are required to operate. At Sugarloaf Dam the net head for the present height of dam varies between 60 and 115 ft.; a turbine with a satisfactory efficiency curve over this range should be obtainable. After the reservoir is raised to its ultimate stage, the net head will vary from 60 to 165 ft., and this will probably involve changing the runners twice each year, with the varying head.

A. J. Wiley,* M. Am. Soc. C. E. (by letter).—Mr. Bryan has presented very ably and fairly some new problems that have arisen in connection with the extensive series of reservoirs that are now being investigated or constructed to serve the San Joaquin Valley, California. The writer's familiarity with these projects impels him to join briefly in their discussion and to express his appreciation of Mr. Bryan's clear and forceful paper.

The physical conditions in the drainage areas tributary to the San Joaquin River not only require the construction of high dams for storage reservoirs, but also make the design and building of such dams practicable. The three principal tributaries of the San Joaquin on which storage development has been initiated are the Tuolumne, the Stanislaus, and the Merced Rivers. These

* Cons. Hydr. Engr., Boise, Idaho.

streams drain the rugged western slope of the Sierra Nevada Mountains, and flow with rapid fall through a succession of canyons and small valleys until they emerge upon the level plains of the San Joaquin Valley.

There are no level bottom-lands or meadows along the course of these streams for favorable reservoir sites. The reservoirs have to be located where the sides of a valley draw in to form a dam site close below a valley section in which the entrance of side streams increases the back-water area.

The narrow channels at the dam sites are carved out of bed-rock of flint-like hardness, and the canyon walls slope steeply upward for hundreds of feet above the stream bed. These conditions make it possible in some cases to use a high arch dam of less than gravity section, and in others to design a gravity section in which upward pressure can be reduced by drainage and offset by arch action.

Because of the extreme hardness of the dioritic bed-rock, the foundation is capable of sustaining extreme stresses and local conditions favor the making of strong concrete of high specific gravity at a low cost.

The part of the Sierras drained by the three rivers was the scene of the greatest placer mining activity the world has ever known, and before the passage of the "Anti-Débris" Law, millions of tons of gravel from the hydraulic mines were carried through the swift and confined canyon sections to raise the river beds and cover the adjacent bottom-lands of the San Joaquin Valley with an inexhaustible supply of the strongest and densest concrete material. Although the gravel in its native beds, and also in the enormous "dumps" or "tailings" adjacent to the old placers, contains a quite serious percentage of "rotten" gravel, apparently sound, but disintegrating on exposure and breaking under a light blow, none but the soundest gravel survived the grinding of the trip down the river. In addition to the sand and gravel of concrete size it carries about 25% of "cobbles" ranging from 3 in. to 8 in. in diameter. The use of these "cobbles" in concrete is probably the most important development since the introduction of the machine mixer. They are passed through the mixer in measured proportions with the other aggregates and make from 20 to 25% of the concrete, with an almost proportionate decrease in cost and a decided increase in strength and weight. The concrete in the Don Pedro Dam on the Tuolumne River required an average of 0.89 bbl. per cu. yd. of cement to make a concrete testing, in 6 by 9-in. cylinders, about 1 800 lb. at 90 days and 2 000 lb. at 150 days, the average cost, including equipment and overhead, being $7.39 per cu. yd.

It appears to the writer that the increase in height of concrete dams in recent years has been made possible by the discovery of favorable dam sites in narrow canyons, and by the improvement in the quality of concrete, with the resulting use of higher limiting stresses, rather than by any special changes in the theory of design.

The development of power as a by-product of irrigation storage is in many cases doubly desirable because of the destruction by the reservoir of an existing power plant, railway, or other public utility, and it seems good public policy to encourage the full development of any new use to offset the loss and

help pay for what is destroyed. Whether or not hydro-electric power shall be developed in connection with a storage dam must be determined by the conditions prevailing at each site. In some cases the cost of storage without the revenue from the by-product power would be too great a burden for the land. Evidently, the development of power for which there was no market would make the storage still less feasible, and where there are so many agencies interested in any development financed by bond issues, there would seem to be no possibility of any such mistake. If the new enterprise cannot make some arrangement with a public utility company for the sale of its power, it would have to present convincing evidence both to the State Bonding Commission and to the financial agencies to justify the issue and sale of bonds for power development.

The cost of power development in connection with storage is usually very low. There is a mistake, however, in the quotation by the author of the cost of the Don Pedro Power Plant, probably arising from his using the Turlock District bond issue alone as the entire cost. The actual cost for 15 000 kv-a. of nominal and 18 000 kw. of actual installed capacity with tail-race capacity and intake provision for 12 000 kw. more, was $911 184.59, or about $60 per kv-a. nominal and $50 per kw. of actual capacity at the switchboard. This low cost is due largely to the construction of the power-house as a part of the dam and the elimination of long penstocks and surge tanks.

Storage in the San Joaquin Valley usually follows the application of the normal flow of the streams through a completed distribution system. The area is highly developed and planted to valuable crops, including orchards and vineyards. To protect these crops in seasons of short supply, resort is made to pumping from wells using either gas or electricity for power. Large areas of the older irrigated sections require drainage to lower the water-table. The most convenient and effective method of drainage is by the use of pumps, which are also operated by gas or electric power. These two uses make a heavy demand for power which synchronizes with the irrigation season and furnishes a market within the project for power developed from irrigation storage. If this market were supplied electrically, it would require a system of secondary power lines over a large part of the district. This system could also deliver power for domestic use on the farms. When interest and depreciation charges are considered, it would not be economically feasible, in many cases, to supply power, except in the more congested areas, and this, naturally and rightly, would be the policy of a public utility company serving such a territory; but where an irrigation district, developing its own power as a by-product, is willing and financially able to stand the burden of extending and maintaining its power lines over non-productive as well as productive territory there is the same right and reason for such action as for the other undisputed functions which it possesses. (See page 921.) As a rule, however, it is probably better for irrigation districts to sell their by-product power wholesale to public utility companies rather than to extend their activities to such a highly technical field, but in the single case of district distribution with which the writer is familiar, the same principles of accounting are followed and the

same insurance to customers is given as are practiced by the competing public utility companies.

EVERETT N. BRYAN,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper had as its origin certain studies by the writer made on behalf of a contemplated irrigation development in the San Joaquin Valley, California. The power requirement of this development and the proximity of the sources of by-product power at the Don Pedro, Melones, and Exchequer storage dams gave rise, in the writer's mind, to the hope that some plan of co-operative development might be evolved wherein those agricultural areas requiring power for irrigation pumping would assist in the construction of storage works to serve those agricultural areas requiring stored water to supplement the natural flow in the late summer and fall. The thought was that the construction of such combination storage and by-product power works might be facilitated if there was co-ordinated effort between the two irrigation interests which would use not only the stored waters but also a considerable block of the by-product power as well.

This thought fascinated the writer, and following an address on the subject before the Sacramento Section of the Society on May 20, 1924, it was suggested that the matter be presented to the Society for publication. This suggestion was accepted in the hope that such a paper would induce a general discussion of the relation of combination irrigation storage and by-product power developments by others who were familiar with the history and general aspects of the problem.

The paper has not developed the general discussion that the writer had expected. There is, however, an interesting paper entitled, "Power in Its Relation to Irrigation", submitted by Henry J. Pierce, President of the Washingon Irrigation and Development Company, and Edward C. Bebb, Assoc. M. Am. Soc. C. E., at The World Power Conference, London, England, in July, 1924, which is recommended to those who wish to pursue the subject further.

The comment by Mr. Wiley is gratefully acknowledged. It is regretted that he has not found time to contribute at greater length from his wide experience. The cost estimate of the power development at Don Pedro given in the paper, was quoted from a preliminary estimate of the District. The figure, $911 184.59, given by Mr. Wiley, has been verified from the reports of actual cost and represents the cost of an installation of three units of 5 000 kv-a. nominal, or 6 000 kv-a. actual, capacity, together with power outlets through the dam for two additional units and a part of the power-house foundation and draft-tube preparation connected therewith.

Mr. Wiley considers that the increase during recent years in the height of concrete dams has been made possible by the improvement in the quality of concrete, with the resulting use of higher limiting stresses, rather than by any special changes in the theory of design. As contributing causes of this increase in the height of dams there might also be added the confidence born

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of successful accomplishment and the necessity that has arisen for larger developments, coupled with the availability of the requisite larger blocks of capital—the simpler, smaller, and less expensive developments having, for the most part, already been made.

The writer is interested in Mr. Guy's description of a typical by-product development in the State of Victoria, Australia. Of particular interest was his showing of the State's intention to establish auxiliary high-head plants. It has been the writer's thought that in connection with the larger by-product power installations at the lower elevations on California streams there should be established auxiliary high-head plants with limited storage on the upper reaches of the same streams.

This course was considered by the Modesto Irrigation District on the Tuolumne River and water filings for such auxiliary plants have already been made by the Pine Flat Water Storage District on Kings River in connection with the proposed Pine Flat Reservoir and by-product power plant. It is felt that such a development is peculiarly adapted to the Pine Flat Project because of the necessity of conserving all the water of the stream in normal years. The water could be passed through the upper auxiliary plants during the non-irrigating season in order to meet the instant demand for power and could then be caught and stored by the large irrigation reservoir below.

Since the publication of the paper the complete details of the contract providing for the construction of the Melones Project have become known. This contract was signed on January 15, 1925, and provides for joint participation by the Pacific Gas and Electric Company on the one hand and the Oakdale and South San Joaquin Irrigation Districts on the other in the development of this project.

The physical and general features of the project are described in the paper.* Under the terms of the contract the two Irrigation Districts will secure all necessary rights of way for the reservoir, power conduit, and power station, all necessary Federal and State permits, and complete the dam to a capacity of 112 500 acre-ft. on or before June 30, 1927. The Power Company will construct the power conduit and station.

The river has an average annual discharge of 1 380 000 acre-ft. and during the period from March 1 to October 31 of each year the Irrigation Districts will have full control of the storage and release of water, except that they may not release water at a rate to exceed 1 700 sec-ft., or store water when there is less than 1 000 sec-ft. of natural flow, and that, in so far as they may do so without prejudice to their own interests, they will be governed by the wishes and needs of the Power Company. During the period from October 31 to March 1, the Power Company will control in the matter of storage and release of waters from the reservoir, except that 36 000 acre-ft. for storage in the foothill reservoir of one of the Districts shall be released to that District, plus such a quantity as is needed by the Districts for immediate irrigation and domestic use. Further, the interests of the Districts

* See p. 917.

will be respected in so far as may be without prejudice to the interests of the Power Company.

There is reserved to the Power Company the exclusive right to control the passage of waters released from its power storage reservoirs on the upper water-shed. One interesting feature of the agreement is the acceptance of the findings of the Division of Water Rights, State Department of Public Works of California, in the matter of water rights on the Stanislaus River, which findings had theretofore been contested by both the Districts and the Power Company.

The water level in the reservoir will not be drawn below an elevation of 640 ft., which gives a minimum static head of 140 ft. for the development of power. A maximum capacity of 30 000 kw. is contemplated, but the Power Company apparently has reserved to itself the right to increase the installation if that is found to be expedient. The Power Company will pay to the Districts in equal semi-annual installments during the next 40 years a total of $5 175 000; at the end of that period payments by the Power Company will cease. Prior to final payment on the contract the Districts are to operate and maintain the storage dam at their own cost and expense, and subsequent thereto such cost will be borne equally by the Districts and the Power Company.

On May 18, 1925, the matter of voting $1 100 000 in each of the two Districts for the construction of the Melones storage works was submitted to the electors and carried by an overwhelming majority. If these bonds are sold on a basis of 5%, the annual cost of the Melones Project to the two Districts will be $110 000 and the assured income by virtue of the contract referred to will be $129 375.

Although the contract is not as favorable to the Districts as the first reports indicated, it does permit the Districts to develop an adequate supply of stored water at a cost scarcely dreamed of a few years ago, before the possibilities of by-product power had been fully explored.

An interesting feature of the plan of development adopted is that the two Irrigation Districts seeking a supply of stored waters should have turned for assistance in financing their project to a power company rather than to another agricultural area seeking power for pumping. By so doing they have unquestionably reduced the immediate cost to themselves and sacrificed somewhat in the matter of ultimate cost. It is the writer's expectation that several years will be required to distribute the large blocks of by-product power which will be developed in California into the general power market and that during this period the price of such by-product power will be uncertain and may decline somewhat. Ultimately, however, this class of power will be fully absorbed by the general power market and there will be a demand for a larger amount than can be developed. At that time the price for such power will have advanced far beyond present rates and those irrigation districts which retain the ownership of their by-product output will be in a position to take advantage of the increased rate.

Looking at the problem from another angle it may be well if the ownership of these sources of power shall have fallen before that time into other hands than those of the irrigation districts developing the storage works, because by such a change in ownership the benefit of these relatively cheap sources of power generated by use of the waters of the State can be passed on to the general public which utilizes the power rather than be held to the sole benefit of the single area which was so fortunately situated as first to be able to appropriate the necessary water. This thought, however, leads into a larger field, which it was not the purpose of this paper to explore, namely, the extent to which present irrigation development should be subsidized through utilization of the power privileges attached to the public water of the State.

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Paper No. 1568

DESIGN OF SYMMETRICAL CONCRETE ARCHES*

By Charles S. Whitney,† M. Am. Soc. C. E.

With Discussion by Messrs. William Cain, Carl B. Andrews, R. R. Martel, E. H. Harder, Charles W. Comstock, A. G. Hayden, H. E. Warrington, Rowland C. Robin, Hardy Cross, Jamison Vawter, and Charles S. Whitney.

Synopsis

The method of analysis here given is believed to be considerably simpler than any accurate method of applying the elastic theory to symmetrical arches heretofore published.

The formulas are derived by actual integration and the results are therefore even more exact than those obtained by the usual method of dividing the arch rib into finite lengths. In order that the integration could be performed, it was necessary to express the arch axis and the thickness of rib mathematically. An equation has been derived which, for different values of the ratio of the loading per unit length of rib at the springing line to that at the crown, gives a curve that has been found to coincide almost exactly with the dead load pressure line for practical bridge arches. Another equation defines very satisfactorily the variation of thickness of the rib from springing line to crown between any desired limits. By substituting these equations in the general elastic equations for symmetrical arches, definite formulas for reactions have been obtained.

Tables are given for the solution of the equations for the form of the axis and rib, and also tables and diagrams for obtaining the reactions due to the dead and live loads and to the temperature and rib-shortening effects, together

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with complete influence lines for the moments at the crown, springing line, and quarter-points, and diagrams giving the maximum and minimum bending moments with corresponding thrusts for the crown, springing line, and quarter-points. These tables and diagrams eliminate all involved calculations such as are necessary in applying the ordinary methods of designing symmetrical arches.

In order that the treatment of this method may be complete and the notation entirely clear, it has been thought best to develop the formulas from the fundamental elastic relations. The graphical construction of the influence lines has also been given completely for both vertical and horizontal loads as it shows the curve for which the equations are derived and serves as a satisfactory method for checking.

The equations for the arch axis and the variation of the rib thickness and the method of integrating them are due to Strassner.* His general method of solution of the arch differs somewhat from that of the writer, and he does not obtain the equations for the V and M' influence lines as given herein.

This quick method of obtaining the stresses in an arch rib makes it possible to proportion the rib more accurately and efficiently for the desired stresses than would be possible by ordinary means. The results given by the tables and diagrams may be used for the final design since they are as accurate as can be obtained by any other method of applying the elastic theory.

For the convenience of those who may desire to use the tables and diagrams for designing, the notation is summarized at the conclusion of the descriptive part of the paper.

Introduction

Although the hingeless concrete or masonry arch is one of the oldest structural forms and has become in recent years one of the most important, a great many arches have not been scientifically or economically designed because of a lack of a simple method of analysis. It has been quite customary to assume the proportions of the rib with the guidance of previous experience or of empirical or semi-empirical formulas for rib thickness. In many cases, the "design" has consisted merely in checking the stresses in an assumed rib. If the stresses came within the limits set by the specification, the design was considered satisfactory. The economy of a design made in this way depends entirely on the skill and experience of the designer, because, as will be shown later, the rib thickness may vary between wide limits and still satisfy the specification.

None of the formulas for rib thickness in use at the present time is of general application, and no general formula can be written which would be simple enough for practical use and still give accurate results for the full range of the variables occurring in practice. In order to determine the stresses in a hingeless arch, it is first necessary to assume a design. In order to determine the effect of a variation in rib thickness on the stresses so

* "Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogenträger."

that the full strength of the materials may be utilized, it is necessary to analyze several assumed designs. On account of the great amount of time and labor involved, this has not ordinarily been possible. Approximate methods have been in use which permit a preliminary study of the rib proportions, but they are not accurate for all cases and are not in general satisfactory for final design.*

The writer believes that the methods given herein will satisfy the need for a simplified, accurate treatment of arch ribs. He has attempted to eliminate all unnecessary details in the application of the elastic theory to the problem and has not mentioned the ellipse of elasticity, because it does not seem to be needed for the treatment of the symmetrical rib. The data given in the tables and diagrams can be used without a thorough knowledge of the entire theory, but the designer should have at least a general understanding of the action of arches.

By far the majority of arches are symmetrical, but the following methods can also be applied to unsymmetrical arches in many cases, as explained by Carl B. Andrews, Assoc. M. Am. Soc. C. E.†

I.—General Elastic Theory

The stresses in an arch rib that is fixed at each end are statically indeterminate. The ordinary conditions of statical equilibrium which give the equations, $\Sigma X = 0$, $\Sigma Y = 0$, and $\Sigma M = 0$, are not sufficient to determine the reactions at the abutments, and it is necessary to write additional equations based on what is known or assumed in regard to the elastic deformations of the rib. As the elastic deformations depend on the design of the rib, it is necessary to assume a definite form for the rib before the forces can be determined. For any particular rib as designed, equations can be derived giving the horizontal, vertical, and angular displacements of one of the abutments, relative to the other, for any desired condition of loading, and the solution of these equations will determine the unknown reactions at the abutments, after which the problem becomes one of simple statics. For ribs with rigidly fixed ends, the displacements of the abutments are zero, and for ribs with elastic supports, these displacements are equal to the yielding of the supports.

General equations for the deflection of curved elastic ribs will first be derived and then applied to the particular case of the symmetrical fixed arch.

Maxwell's Theorem.—One of the fundamental relations which is of great value in deriving formulas for the solution of elastic ribs is expressed by Maxwell's theorem of reciprocal displacements, the derivation of which is given here in order that its meaning may be fully understood.

Let P_1, P_2, P_3, etc., be forces acting on an elastic body and Δ_1, Δ_2, Δ_3, etc., be the total displacements of the points of application of the forces in the direction of the respective forces. If the forces are applied gradually

* Such a method is given by Victor H. Cochrane, M. Am. Soc. C. E., in his paper, "Design of Symmetrical Hingeless Concrete Arches", *Proceedings*, Engrs. Soc. of Western Pennsylvania (November, 1916), p. 647, and also by J. A. L. Waddell, M. Am. Soc. C. E., in his "Bridge Engineering," Vol. I, p. 868.

† "Reactions for a Particular Type of Unsymmetrical Arch", *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 438.

and simultaneously, they will vary in magnitude from zero to P as the displacements vary from zero to Δ. As the forces must vary directly as the displacement if the elastic limit is not exceeded, the average force will be $\frac{1}{2} P$, exerted through a distance, Δ. Therefore, the total external work done by the forces during the time of their application will be:

$$W = \frac{1}{2} P_1 \Delta_1 + \frac{1}{2} P_2 \Delta_2 + \frac{1}{2} P_3 \Delta_3 + \text{etc.}$$

This equation may be written:

$$W = \frac{1}{2} P_a \Delta_a + \frac{1}{2} P_b \Delta_b$$

in which, $P_a \Delta_a$ and $P_b \Delta_b$ each can be equal to a single force multiplied by its corresponding displacements if there are only two forces to be considered or it can be equal to the summation of a group of forces each multiplied by its displacement. In the latter case, it can express the effect of a couple as well as of a single force.

For example, in Fig. 1 let the two forces, P_1 and P_2, produce the deformations, Δ_1 and Δ_2, when acting simultaneously. If these forces are equal and opposite, they form a couple the moment of which is:

$$M = P_1 l = P_2 l; \text{ or, } P_1 = \frac{M}{l}$$

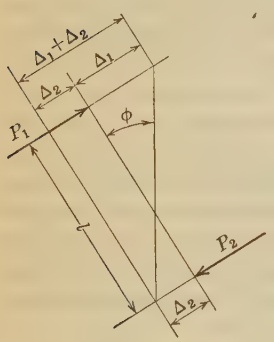

Fig. 1.

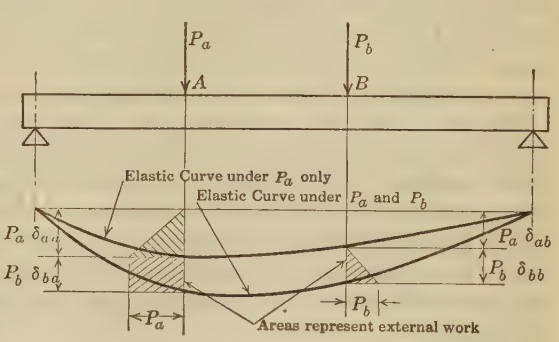

Fig. 2.

The angular displacement, ϕ, is equal to:

$$\frac{\Delta_1 + \Delta_2}{l}; \text{ or, } \Delta_1 + \Delta_2 = \phi\, l^*$$

The work done by these two forces is:

$$W = \frac{1}{2} P_1 \Delta_1 + \frac{1}{2} P_2 \Delta_2 = \frac{1}{2} P_1 (\Delta_1 + \Delta_2)$$

Substituting the values given previously:

$$W = \frac{1}{2} \cdot \frac{M}{l} \cdot \phi\, l = \frac{1}{2} M \phi\,.$$

* This holds true only for small angular displacements such as in arches.

Therefore, the term, $\frac{1}{2} P_a \Delta_a$, may be used for a couple as well as for a single force and, in the case of a couple, P_a represents the moment of the couple and Δ_a, the corresponding angular displacement. P_a and P_b will be mentioned as single forces, but as has been proven they can represent either a single force or the moment of a couple.

Let δ_{aa} be the displacement of the point of application of P_a in the direction of the line of action of P_a, produced by a unit value of P_a acting alone. Let δ_{ab} represent the displacement at the point of application of the force, P_b, in the direction of P_b, produced by a unit value of P_a acting alone. Also, δ_{bb} and δ_{ba} will be the corresponding displacements of the points, B and A, respectively, produced by a unit value of P_b acting alone. Then,

$$\Delta_a = P_a \delta_{aa} + P_b \delta_{ba}$$
$$\Delta_b = P_a \delta_{ab} + P_b \delta_{bb}$$

If first the load, P_a, is placed on the structure alone, the displacement at A will be $P_a \delta_{aa}$ and the work done by P_a will be $\frac{1}{2} P_a^2 \delta_{aa}$. If P_b is then added, the additional displacement of A will be $P_b \delta_{ba}$ and the additional displacement of B will be $P_b \delta_{bb}$. The force, P_a, will be acting in its full amount through the distance, $P_b \delta_{ba}$, and, therefore, the work done when P_b is added, will be:

$$P_a P_b \delta_{ba} + \frac{1}{2} P_b^2 \delta_{bb}$$

or the total amount of work done by both forces is:

$$W = \frac{1}{2} P_a^2 \delta_{aa} + \frac{1}{2} P_b^2 \delta_{bb} + P_a P_b \delta_{ba}$$

This relation is illustrated by Fig. 2 which represents two loads on a simple beam. This is not a general case, but is used for simplicity.

If it be assumed that P_b be placed on the structure first, and then P_a added, the equation for the work done becomes:

$$W = \frac{1}{2} P_a^2 \delta_{aa} + \frac{1}{2} P_b^2 \delta_{bb} + P_a P_b \delta_{ab}$$

As the total work in each case must be the same, it follows that:

$$P_a P_b \delta_{ab} = P_a P_b \delta_{ba}$$

or,

$$\delta_{ab} = \delta_{ba}$$

Maxwell's theorem is based on this equation. For an arch rib, it may be expressed as follows:

1.—If two unit forces, P_a and P_b, act separately at any two points, A and B, respectively, of an elastic rib, the linear displacement of the point, A, in the direction parallel to the line of action of P_a produced by the force, P_b, is the same as the linear displacement of the point, B, in the direction parallel to the line of action of the force, P_b, produced by the force, P_a.

2.—If an elastic rib is acted on separately by a unit force, P_a, at any point, A, and a unit couple, M_b, at any point, B, the angular displacement in radians of the tangent to the rib at B produced by the force, P_a, is equal to the linear displacement of the point, A, in the direction parallel to the line of action of P_a produced by the couple, M_b.

3.—If two unit couples, M_a and M_b, act separately at any two points, A and B, respectively, of an elastic rib, the angular displacement of the tangent to the rib at A produced by the couple, M_b, is equal to the angular displacement of the tangent to the rib at B produced by the couple, M_a.

General Equations for Elastic Curve for Arch Ribs.—In order to derive the general equation for arch ribs, consider a length, ds, of a curved concrete rib, as shown in Fig. 3. The section of the rib is assumed to be rectangular and symmetrical. The following notation will be used in addition to that shown in the diagram:

T = direct thrust;
M = bending moment;
A = area of section of rib
$= \{A_c + (n - 1) A_s\}$ if reinforced with steel, in which,
A_c = area of concrete,
A_s = area of steel,
$n = \dfrac{\text{modulus of elasticity of steel}}{\text{modulus of elasticity of concrete}}$;
I = moment of inertia of the rib
$= I_{(\text{concrete})} + n\, I_{(\text{steel})}$;
f = unit stress at the center of the rib;
f' = unit stress at the outer fiber; and
E = modulus of elasticity of concrete.

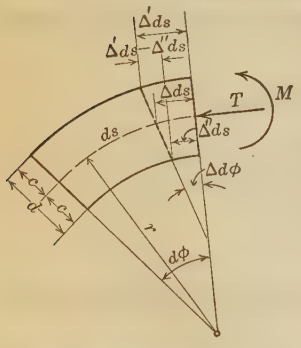

Fig. 3.

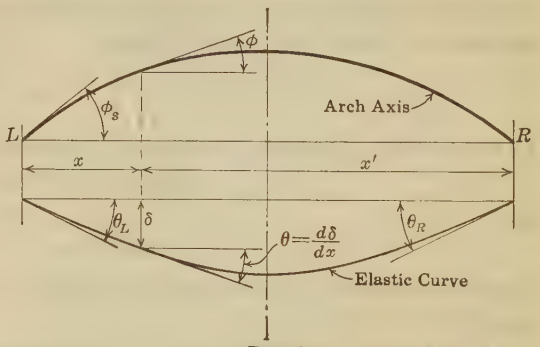

Fig. 4.

The stress at the center of the rib is $f = \dfrac{T}{A}$. The stress at the outer fiber is $f' = \dfrac{T}{A} + \dfrac{M\,c}{I}$. The length of the center fiber being $d\,s$, the length of the

outer fiber is $\frac{r+c}{r}\,ds$, and that of the inner fiber, $\frac{r-c}{r}\,ds$. Then, the deformations being equal to the unit stress multiplied by the length divided by the modulus of elasticity,

$$\Delta' ds = \frac{f' ds}{E} \cdot \frac{r+c}{r} = \left(\frac{T}{A} + \frac{Mc}{I}\right) \frac{r+c}{r} \cdot \frac{ds}{E}$$

and,

$$\Delta'' ds = \left(\frac{T}{A} - \frac{Mc}{I}\right) \cdot \frac{r-c}{r} \cdot \frac{ds}{E}$$

From Fig. 3, the angular deformation, $\Delta d\phi$, of the rib in the length, ds, is given by:

$$\Delta d\phi = \frac{\Delta' ds - \Delta'' ds}{2c}$$

But,

$$\Delta' ds - \Delta'' ds = \left[\frac{T}{A}\left(\frac{r+c}{r} - \frac{r-c}{r}\right) + \frac{Mc}{I}\left(\frac{r+c}{r} + \frac{r-c}{r}\right)\right] \frac{ds}{E}$$
$$= \left(\frac{T}{A}\frac{2c}{r} + \frac{2Mc}{I}\right) \frac{ds}{E}$$

Hence,

$$\Delta d\phi = \frac{T ds}{EAr} + \frac{M ds}{EI} \quad \ldots\ldots (1)$$

The value of

$$\Delta ds = \frac{\Delta' ds + \Delta'' ds}{2}$$

Substituting the value for $\Delta' ds$ and $\Delta'' ds$, Equation (1) becomes:

$$\Delta ds = \frac{T ds}{EA} + \frac{c^2}{r} \cdot \frac{M ds}{EI} \quad \ldots\ldots (2)$$

For the proportions of arch ribs used in practice, Equations (1) and (2) can be written, as follows, without appreciable error:

$$\Delta d\phi = \frac{M ds}{EI} \quad \ldots\ldots (3)$$

$$\Delta ds = \frac{T ds}{EA} \quad \ldots\ldots (4)$$

Displacements Due to Bending.—If ϕ is the angle between the tangent to the rib axis at any point and the x-axis, which is the chord of the rib, $dx = \cos\phi\, ds$.

Let δ be the ordinate at any point of the elastic curve for a given loading. That is, δ is the vertical displacement of the corresponding point of the rib. Then, the slope of the elastic curve, which is $\frac{d\delta}{dx}$, is the angular displacement of the tangent to the rib axis and will be called θ. The derivative of this angular displacement, θ, with respect to x, is equal to $\frac{\Delta d\phi}{dx}$, the value of which

is given by Equation (3). The value, $\Delta d\phi$, is the increase in the slope angle of the elastic curve for the length, ds, and, therefore,

$$\frac{d\theta}{ds} = \frac{\Delta d\phi}{ds} = -\frac{M}{EI}$$

The negative sign is introduced because the value of θ is measured from the chord instead of from the tangent as in the derivation of Equation (3). Then, as

$$ds = \frac{dx}{\cos\phi}$$

$$\frac{d\theta}{dx} = -\frac{M}{EI\cos\phi}$$

and,

$$\frac{d^2\delta}{dx^2} = \frac{d\theta}{dx} = -\frac{M}{EI\cos\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

This differential equation for the elastic line can be used for determining the deflection under any loading by integrating twice and introducing the proper constants of integration. The sign should be changed if the deflection relative to a tangent instead of to the chord is desired.

Equation (5) has also another important meaning. For a beam under vertical loading, the first derivative of the bending moment is the shear, or $\frac{dM}{dx} = V$, and the first derivative of the shear is the load per unit length of beam, w_u, or,

$$\frac{dV}{dx} = w_u = \frac{d^2M}{dx^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

From Equation (5) it is seen that,

$$\frac{d^2\delta}{dx^2} = -\frac{M}{EI\cos\phi}$$

In these two analogous equations the deflection corresponds to the bending moment, M, and the term, $-\frac{M}{EI\cos\phi}$, corresponds to w_u, the load per unit length. It follows, therefore, that:

The deflection, δ, is equal to the bending moment which would be produced in a beam if each partial length, ds, is loaded with its corresponding value of $\frac{M\,dx}{EI\cos\phi}$.

The supports of this imaginary beam are at those points where the arch rib is considered to be supported. The slope of the elastic curve at any point is equal to $\theta = \frac{d\delta}{dx}$, which is obtained by integrating Equation (5) once. Therefore, for a rib supported at its ends:

The slope of the elastic curve at any point is equal to the shear which would be produced in a beam if each partial length, ds, is loaded with its corresponding value of $\frac{M\,dx}{EI\cos\phi}$.

The tangent of the angle of deflection at the ends of the span is equal to the reactions at the corresponding ends of a beam of which each partial length is loaded with its value of $\frac{M\,d\,x}{E\,I\cos\phi}$.

Referring to Fig. 4:

$$\left.\begin{aligned}\theta_L &= \frac{1}{l}\int\frac{M\,x^1\,d\,x}{E\,I\cos\phi}\\ \theta_R &= \frac{1}{l}\int\frac{M\,x\,d\,x}{E\,I\cos\phi}\end{aligned}\right\}\quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

By means of the foregoing rules, the angular or linear displacements of any points in a rib can easily be determined graphically when the bending moments are known. It is merely necessary to divide the rib into sections small enough to give the desired accuracy, to compute the values of $\frac{M\,d\,x}{E\,I\cos\phi}$, or $\frac{M\,d\,s}{E\,I}$, and to construct the bending moment and shear diagrams as if these values were loads applied at the centers of their respective sections. The resulting shear at any point is the angular deflection and the bending moment is the deflection, δ.

It has been assumed that δ is the deflection normal to the x-axis and that the $\frac{M\,d\,s}{E\,I}$ loads are acting also normal to the x-axis. It should be noted that the method applies as well to inclined or horizontal displacements. In any case, the elastic line is to be constructed on an axis perpendicular to the direction of the displacements desired and the $\frac{M\,d\,s}{E\,I}$ loads are then applied normal to this axis.

An example of the application of this graphical method is given by Fig. 5. It is assumed that the form of the rib and the size and location of all the forces acting on it are known. It is desired to construct the elastic line of the rib under this assumed loading. The bending moment diagram is first constructed on an axis, $L' - R'$, perpendicular to the direction of the desired displacements. The rib is then divided into small lengths and the value of $\frac{M\,\Delta\,s}{E\,I}$ is computed for each division. Considering these values of $\frac{M\,\Delta\,s}{E\,I}$ to be vertical forces, lay off the load line of the force polygon and draw rays to any pole point, O, of which the perpendicular distance from the load line is p. The funicular polygon, $L'' - R''$, is constructed with sides parallel to the corresponding rays of the force polygon. This funicular polygon is then the bending moment diagram of the rib, considering each division loaded with its value of $\frac{M\,\Delta\,s}{E\,I}$. The ordinates of this diagram multiplied by the pole length, p, give the displacements, or,

$$\text{Displacement, } \delta = p\,d$$

This displacement can be measured from any tangent or chord of the curve as desired. For instance, if the rib is fixed at the right end, the displacement

of the point, A, would be $p\,d_2$ measured from the tangent to the curve at the right end, $L_1'' - R''$. If the displacement relative to the line joining the two ends is desired, it would be $p\,d_1$.

The angular displacement of the tangent at any point relative to any other tangent or chord can also be determined from the linear displacements. For example, as shown in Fig. 5, the angular displacement, θ_L, between the tangent at the left end and the line through the two ends is:

$$\theta_L = \frac{p\ (R'' - R_1'')}{l}$$

By extending the sides of the funicular polygon to d_2 (Fig. 5), it may be seen from similar triangles in the force and funicular polygons that,

$$d_2 = \frac{x_6}{p} \cdot \frac{M_6\ \Delta\ s_6}{E\ I_6} + \frac{x_5}{p} \cdot \frac{M_5\ \Delta\ s_5}{E\ I_5} +, \text{ etc.}, = \frac{1}{p} \sum_A^R \frac{M\ x\ \Delta\ s}{E\ I}$$

or,

$$p\ d_2 = \sum_A^R \frac{M\ x\ \Delta\ s}{E\ I}$$

which, it will be proven, is the displacement, δ.

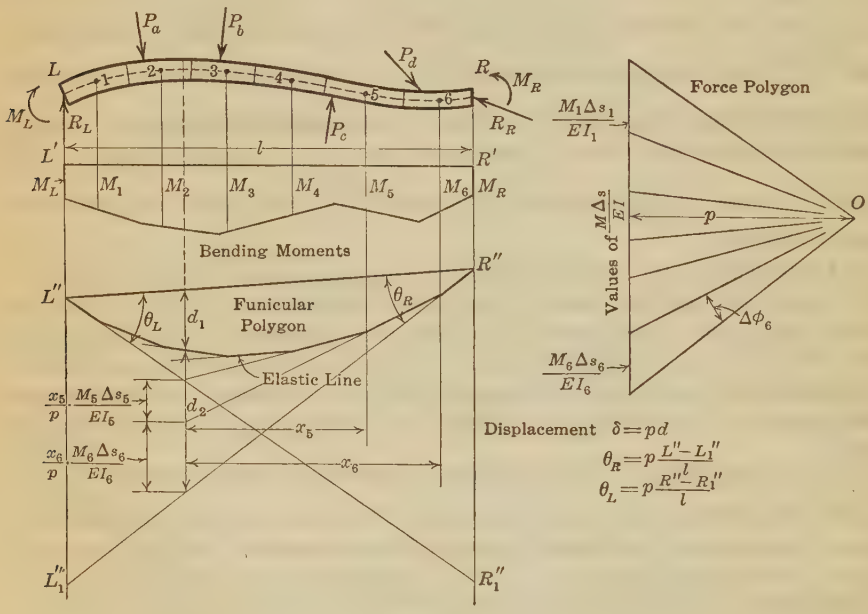

FIG. 5.

Also, it should be noted that the angle between the adjacent sides of the funicular polygon is the same as that between the adjacent rays of the force polygon and equals $\frac{1}{p}\frac{M\ \Delta\ s}{E\ I}$, which is equal to $\frac{\Delta\ \phi}{p}$.

Actually, the elastic curve is not the funicular polygon, which consists of a broken line with straight sides, but rather the smooth curve drawn tangent

to all these sides. For simplicity, the rib in Fig. 5 was divided into only six sections. In order to get the proper accuracy, about twenty sections should be used for arch ribs.

Displacements Relative to a Tangent to the Rib at Any Point Produced by Bending Only.—In the rib shown in Fig. 6, it is desired to find the displacement of the point, A, with respect to the tangent at the point, B. The rib will be assumed as fixed at B. Considering one small length of the rib, ds, which is subjected to a bending moment, M, the angular distortion of this section will be $\Delta d\phi = \frac{M\,ds}{EI}$ (from Equation (3)), and the summation for the entire rib will be:

$$\Delta\phi = \int_A^B \frac{M\,ds}{EI} = \int_A^B M\,dw \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which the elastic weight, $dw = \frac{ds}{EI}$.

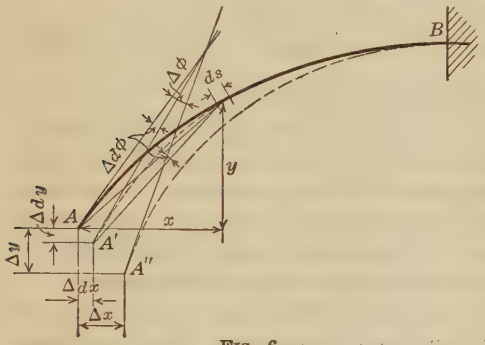

FIG. 6.

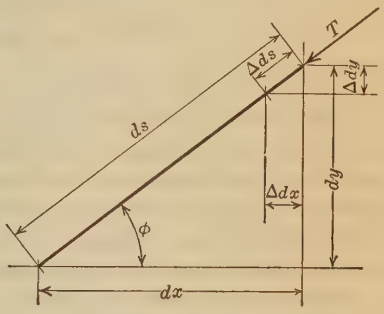

FIG. 7.

The vertical or y-displacement of the point, A, due to the bending of this one small length of rib will be:

$$\Delta dy = x\,\Delta d\phi = \frac{M\,x\,ds}{EI}$$

Or, for the whole rib,

$$\Delta y = \int_A^B \frac{M\,x\,ds}{EI} = \int_A^B M\,x\,dw \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Similarly,

$$\Delta x = \int_A^B \frac{M\,y\,ds}{EI} = \int_A^B M\,y\,dw \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

It should be noted that the origin of the co-ordinates, x and y, is at the moving point, A, and the integration is from the moving point to the fixed point, B.

Displacements Due to the Direct Thrust.—The stress due to direct thrust is $f = \frac{T}{A}$. The change in length of rib for the section which has a length, ds, produced by this direct stress is:

$$\Delta ds = -\frac{T\,ds}{EA}$$

From Fig. 7:

$$\Delta d y = d \delta = \Delta d s \sin \phi$$

Then,

$$\frac{d \delta}{d x} = \frac{\Delta d s \sin \phi}{d x} = -\frac{T d s \sin \phi}{E A d x} = -\frac{T \tan \phi}{E A} \quad \ldots\ldots (11)$$

The deflection, δ, due to the normal thrust can be determined by integrating this equation.

The horizontal displacement will be:

$$\Delta d x = \Delta d s \cos \phi = -\frac{T \cos \phi d s}{E A}$$

Hence,

$$\Delta x = -\int \frac{T \cos \phi d s}{E A} \quad \ldots\ldots (12)$$

The horizontal displacement due to both bending and direct thrust is given by the combination of Equations (10) and (12):

$$\Delta x = \int_A^B \frac{M y d s}{E I} - \int_A^B \frac{T \cos \phi d s}{E A} \quad \ldots\ldots (13)$$

II.—Analysis of Symmetrical Arch Ribs

The analysis here given is one of the simplest and most direct methods possible for determining the influence lines for the reactions, bending moments, and thrusts, under moving loads, for an arch rib with fixed ends. These influence lines make it possible to determine accurately the effects of concentrated loads and to place the loads correctly for maximum stresses at different sections of the rib instead of arbitrarily loading a certain part of the span.

The graphical solution given can be performed quickly and can be made to check the analytical solution within a fraction of 1 per cent. This is certainly accurate enough for practical purposes and has an advantage in that the results can be more readily checked than a set of tabulated values. Even if the analytical solution is used, an understanding of the graphical method gives a better conception of the meaning of the figures so that they do not appear to be merely abstract tables. To avoid errors in individual values it is always well to plot the values obtained analytically to see that they fall on a smooth curve.

Derivation of Equations.—The equations for the reactions of the arch will be derived by means of the formulas given in Section I for the displacements. It is assumed that the rib is symmetrical with the abutments at the same level.

It will first be considered that the left abutment is cut away, Fig. 8, leaving the reactions, H, M_L, and V_L, acting on the rib as a simple cantilever.

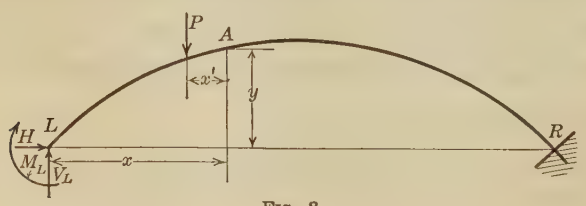

Fig. 8.

The following notation will be used:

M_r = moment of reactions at any section;
M_p = moment of loads at any section;
$\Delta_r x$ = horizontal displacement of the left end due to the reactions;
$\Delta_r y$ = vertical displacement of the left end due to the reactions;
$\Delta_r \phi$ = angular displacement of the left end due to the reactions;
$\Delta_p x$ = horizontal displacement of the left end due to the loads;
$\Delta_p y$ = vertical displacement of the left end due to the loads;
$\Delta_p \phi$ = angular displacement of the left end due to the loads;
$\Delta' x$ = horizontal movement of the left abutment;
$\Delta' y$ = vertical movement of the left abutment; and,
$\Delta' \phi$ = angular movement of the left abutment.

Although the left abutment has been cut away from the rib, leaving that end free to move whereas it was fixed before, there can be no tendency for the rib to move because the same abutment forces are still acting on it. It will be seen that the sum of the displacements of the left end of the cantilever produced by the loads and the reactions must be equal to the movement of the abutment before it was cut away. If the rib is rigidly fixed at the abutments, this movement is zero. The following equations can be written:

$$\left.\begin{aligned} \Delta_r x + \Delta_p x &= \Delta' x \\ \Delta_r y + \Delta_p y &= \Delta' y \\ \Delta_r \phi + \Delta_p \phi &= \Delta' \phi \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Using the left end of the rib as the origin of co-ordinates, the moment, M_r, at any point, A, due to the reactions is:

$$M_r = M_L + V_L x - H y \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

Substituting this value in Equations (8), (9), and (10) from Section I,

$$\left.\begin{aligned} \Delta_r x &= \int_L^R \frac{M_r y\, d s}{E I} = \int_L^R (M_L y + V_L x y - H y^2)\, dw \\ \Delta_r y &= \int_L^R \frac{M_r x\, d s}{E I} = \int_L^R (M_L x + V_L x^2 - H x y)\, dw \\ \Delta_r \phi &= \int_L^R \frac{M_r\, d s}{E I} = \int_L^R (M_L + V_L x - H y)\, dw \end{aligned}\right\} \quad \ldots\ldots (16)$$

As before, $d w = \dfrac{d s}{E I}$.

If the center of co-ordinates is now shifted to the center of gravity of the values of $\dfrac{ds}{E I}$, the terms, $\displaystyle\int_L^R \frac{y\, d s}{E I}$, $\displaystyle\int_L^R \frac{x\, d s}{E I}$, and $\displaystyle\int_L^R \frac{x y\, d s}{E I}$, will be equal to zero. This condition is shown in Fig. 9. The reactions, H and V_L, are considered to be acting on rigid cantilever arms attached to the left end so that they are acting through the "elastic center of gravity." In order that the moment at the left end shall be kept the same as before, it is necessary to replace M_L, with the moment, M', which must be equal to $M_L - H b + \dfrac{V_L l}{2}$, or,

$$M_L = M' + H b - \frac{V_L l}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

This shift in co-ordinates can be effected by substituting in Equation (15), $(y + b)$ for y and $\left(x + \frac{l}{2}\right)$ for x. Then,

$$M_r = M' + V_L\, x - H\, y$$

and Equations (16) become:

$$\Delta_r x = \int_L^R (M'\, y + V_L\, x\, y - H\, y^2)\, d\, w$$

$$\Delta_r y = \int_L^R (M'\, x + V_L\, x^2 - H\, x\, y)\, d\, w$$

$$\Delta_r \phi = \int_L^R (M' + V_L\, x - H\, y)\, d\, w$$

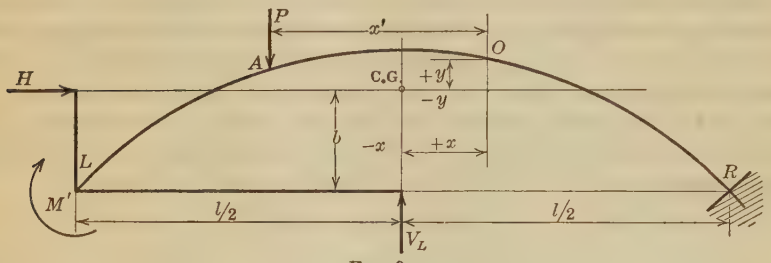

FIG. 9.

Then, as,

$$\int_L^R x\, d\, w = \int_L^R y\, d\, w = \int_L^R x\, y\, d\, w = 0$$

these equations reduce to the following:

$$\Delta_r x = -\int_L^R H\, y^2\, d\, w$$

$$\Delta_r y = \int_L^R V_L\, x^2\, d\, w$$

$$\Delta_r \phi = \int_L^R M'\, d\, w$$

giving the displacements produced by bending only without considering the effect of direct thrust, which in the case of the vertical reaction is so slight as to be negligible. For flat arches, however, the effect of the horizontal thrust should be considered. The values of $\Delta_r x$, $\Delta_p x$, and $\Delta' x$, are considered positive when the span is lengthened and negative when it is shortened.

From Equation (12) for direct thrust,

$$\Delta x = -\int \frac{T \cos \phi\, d\, s}{E\, A}$$

As $T = H \cos \phi$, and $d\, s = \dfrac{d\, x}{\cos \phi}$,

$$\Delta x = -\int \frac{H \cos \phi\, d\, x}{E\, A}$$

Then the displacements due to bending and thrust at the reaction can be written:

$$\left.\begin{aligned} \Delta_r x &= -H\left(\int_L^R y^2\, d\,w + \int_L^R \frac{\cos\phi\, d\,x}{E\,A}\right) \\ \Delta_r y &= V_L \int_L^R x^2\, d\,w \\ \Delta_r \phi &= M' \int_L^R d\,w \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (18)$$

The moment at any point, O, due to a single superimposed vertical load is equal to $-P\,x'$ (see Fig. 9). The value of x' is always considered positive. Again, using Equations (8), (9), and (10), the displacements due to the load, P, are:

$$\left.\begin{aligned} \Delta_p x &= \int_A^R M_p\, y\, d\,w = -P\int_A^R x'\, y\, d\,w \\ \Delta_p y &= \int_A^R M_p\, x\, d\,w = -P\int_A^R x'\, x\, d\,w \\ \Delta_p \phi &= \int_A^R M_p\, d\,w = -P\int_A^R x'\, d\,w \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (19)$$

Substituting Equations (18) and (19) in Equation (14) and solving for the values of H, V_L, and M',

$$\left.\begin{aligned} H &= \frac{-P\int_A^R x'\, y\, d\,w}{\int_L^R y^2\, d\,w + \int_L^R \frac{\cos\phi\, d\,x}{E\,A}} - \frac{\Delta' x}{\int_L^R y^2\, d\,w + \int_L^R \frac{\cos\phi\, d\,x}{E\,A}} \\ V_L &= \frac{P\int_A^R x'\, x\, d\,w}{\int_L^R x^2\, d\,w} + \frac{\Delta' y}{\int_L^R x^2\, d\,w} \\ M' &= \frac{P\int_A^R x'\, d\,w}{\int_L^R d\,w} + \frac{\Delta' \phi}{\int_L^R d\,w} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots (20)$$

Effect of Rib-Shortening.—When the abutments are rigid, there is no yielding and $\Delta' x$, $\Delta' y$, and $\Delta' \phi$ are zero. The formula for H can be further simplified by separating the effect of direct thrust or rib-shortening from the effect of bending. With $\Delta' x = 0$,

$$H = \frac{-P\int_A^R x'\, y\, d\,w}{\int_L^R y^2\, d\,w + \int_L^R \frac{\cos\phi\, d\,x}{E\,A}} = \frac{-P\int_A^R x'\, y\, d\,w}{\int_L^R y^2\, d\,w \,.\, (1+u)}$$

$$= \frac{-P\int_A^R x'\, y\, d\,w}{\int_L^R y^2\, d\,w} - H\,u \quad \ldots\ldots\ldots\ldots (20a)$$

in which,

$$u = \frac{\int_L^R \frac{\cos \phi \, d x}{E A}}{\int_L^R y^2 \, d w} = \frac{l}{E A_m \int y^2 \, d w}$$

and A_m is the reciprocal of the mean value of $\frac{\cos \phi}{A}$ so that,

$$\frac{l}{E A_m} = \int_L^R \frac{\cos \phi \, d x}{E A}$$

The value of $\int x' y \, d w$ is always negative, hence H is positive.

In Equation (20a), the last term gives the effect of rib-shortening due to the live load and as it is small compared to the effect of bending, it may be calculated separately without appreciable error. The rib-shortening thrust will be called H_{RS} and is, from Equation (20a),

$$H_{RS} = - \frac{H l}{E A_m \int y^2 \, d w} = - H u \quad \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

The rib-shortening effect of the dead load thrust can be derived from the formula:

$$H = \frac{\Delta' x}{\int_L^R y^2 \, d w + \int_L^R \frac{\cos \phi \, d x}{E A}}$$

as the effect of a shortening of the rib is the same as a movement of the left abutment toward the left. If H_d is the horizontal thrust due to the dead load, $T = \frac{H_d}{\cos \phi}$, and from Equation (12):

$$\Delta' x = - \int_L^R \frac{T \, d x}{E A} = - H_d \int_L^R \frac{d x}{E A \cos \phi}$$

Then,

$$H_{RS} = - \frac{H_d \int_L^R \frac{d x}{E A \cos \phi}}{\int_L^R y^2 \, d w + \int_L^R \frac{\cos \phi \, d x}{E A}} = - H_d \frac{u'}{1 + u} \quad \ldots\ldots (21a)$$

in which, u is as before, and,

$$u' = \frac{\int_L^R \frac{d x}{E A \cos \phi}}{\int_L^R y^2 \, d w} = \frac{l}{E A'_m \int y^2 \, d w}$$

with A'_m equal to the reciprocal of the mean value of $\frac{1}{A \cos \phi}$, so that:

$$\frac{l}{E A'_m} = \int_L^R \frac{d x}{E A \cos \phi}$$

For any but extremely flat arches, Equation (21*a*) can be written for all practical purposes, as

$$H_{RS} = -H_a u'$$

The rib-shortening thrust due to both live and dead loads together can be computed quite closely at one time by multiplying the combined H by u', or by $\frac{u'}{1+u}$. This thrust acts along the horizontal axis through the elastic center of gravity of the rib and, therefore, the moment at any point due to the rib-shortening is:

$$M_{RS} = -H_{RS} y = H u' y \quad \ldots\ldots\ldots\ldots\ldots\ldots (21b)$$

If the rib-shortening effect is to be considered separately, the live load thrust for a rib with rigidly fixed ends becomes merely:

$$H = \frac{-P \int_A^R x' y \, d w}{\int_L^R y^2 \, d w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Effect of Change of Temperature.—An expansion of the rib due to a rise in temperature has the same effect as a horizontal movement of the abutment—shortening the span—and a contraction due to a drop in temperature has the effect of lengthening the span. This horizontal movement of the abutment, $\Delta' x$, does not affect the values of V_L and M', but it produces a horizontal thrust given by Equation (20) as:

$$H = -\frac{\Delta' x}{\int_L^R y^2 \, d w + \int_L^R \frac{\cos \phi \, d x}{E A}} = -\frac{\Delta' x}{\int_L^R y^2 \, d w \, . \, (1+u)}$$

as,

$$u = \frac{\int_L^R \frac{\cos \phi \, d x}{E A}}{\int_L^R y^2 \, d w}$$

For a change in temperature of the rib:

$$\Delta' x = -t \alpha l$$

in which,

t = change in temperature, in degrees; and
α = coefficient of thermal expansion.

Then, the horizontal thrust due to a temperature change is:

$$H_t = \frac{t \alpha l}{\int_L^R y^2 \, d w \, . \, (1+u)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23)$$

The value of u is so small that it may be neglected except for extremely flat arches, and the thrust becomes:

$$H = \frac{t \alpha l}{\int_L^R y^2 \, d w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (23a)$$

The bending moment due to the temperature thrust is:

$$M_t = -H_t\, y \qquad (23b)$$

The horizontal thrusts due to both rib-shortening and temperature change follow the horizontal axis through the elastic center of gravity of the rib.

Graphical Analysis.—For the purpose of graphical analysis, the arch rib should be divided into short sections, and the value of $\frac{\Delta s}{E I} = \Delta w$ computed for each section. About twenty divisions will give sufficient accuracy for most cases and may be made with a constant value of Δx, or with a constant value of Δw, according to the preference of the designer. There should be no appreciable difference between the results obtained by the two methods.* The values of Δw as computed are considered to be acting at the centers of their respective divisions, although this is not exactly accurate for tapering ribs.

The computation may be made analytically by summing numerically instead of graphically.

Geographical Determination of H for Vertical Loads.—The equation for H produced by a single vertical load, P, considering the ends of the arch as rigidly fixed and neglecting the effect of rib-shortening, is (Equation (22)):

$$H = P\,\frac{-\int_A^R x'\, y\, d\, w}{\int_L^R y^2\, d\, w} = P\,\frac{-\sum_A^R x'\, y\, \Delta w}{\sum_L^R y^2\, \Delta\, w}$$

or, when Δw has a constant value:

$$H = P\,\frac{-\sum_A^R x'\, y}{\sum_L^R y^2}$$

Assuming the rib to act as a cantilever fixed at the right end as in Fig. 10, with a unit horizontal thrust, $H = 1$, the moment at any point in the rib will be equal to $M = H\, y = y$. The horizontal displacement of the left end where H is applied, will then be:

$$\Delta x = \delta_{HL} = \int_L^R M\, y\, d\, w = \int_L^R y^2\, d\, w\dagger$$

This is the same as the denominator in Equation (22). According to the rule in Section I it can be obtained by drawing the bending-moment diagram for the rib, assuming that each value of $y\, \Delta w$ is a force acting horizontally at the center of its respective division. (Because $\frac{M\, d\, s}{E\, I} = y\, \Delta\, w$.)

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* For the usual method of dividing the arch rib so that the values of $\frac{\Delta s}{E I}$ are equal, see Hool's "Reinforced Concrete Construction", Vol. III, p. 32.

† Where the letter, δ, with sub-letters, is used to denote displacement, the first sub-letter serves to identify the force producing the displacement and the second gives the point at which the displacement is measured. For example, δ_{HA} means the displacement produced by the force, H, measured at the point, A.

If a unit vertical load, $P = 1$, be applied to any point, A, of the rib, the moment produced by P will be $M = -P x' = -x'$ and, therefore, the horizontal displacement of the left end where H is applied, due to P is:

$$\Delta x = \int_L^R M y \, d w = -\int_L^R x' y \, d w$$

This term is the same as the numerator in Equation (22). According to Maxwell's theorem, the horizontal displacement of the point, L', produced by a unit vertical load at A is equal to the vertical displacement of Point A produced by a unit horizontal load at L'.

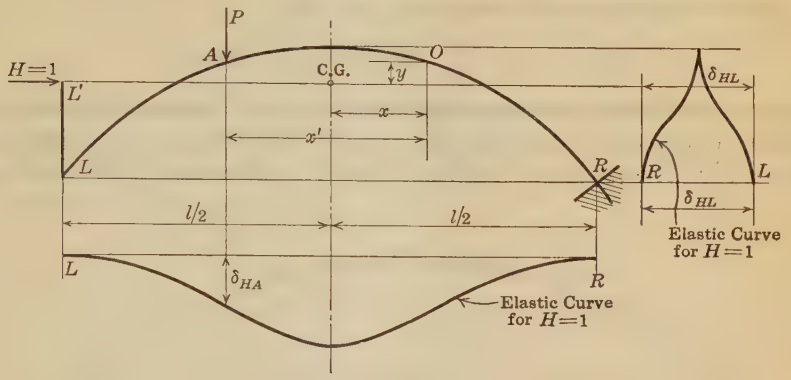


FIG. 10.

Therefore, applying a unit horizontal force, H, at L' as before, the vertical displacement, δ_{HA}, of the point, A, is equal to the horizontal displacement of L' due to $P = 1$, or,

$$\delta_{HA} = -\int_A^R x' y \, d w$$

As $M = y$, the value of δ_{HA} can easily be found graphically by constructing the bending-moment curve for the rib with each section, Δs, loaded with its value of $y \, \Delta w$ acting vertically at the center of the section. The ordinate of this bending-moment curve is δ_{HA}. Then, referring to Fig. 10:

$$H = P \frac{\delta_{HA}}{\delta_{HL}} \qquad (24)$$

The temperature thrust is:

$$H_t = \frac{t \alpha l}{\delta_{HL}} \qquad (25)$$

The rib-shortening thrust is:

$$H_{RS} = \frac{-H l}{E A'_m \delta_{HL}} = -H u' \qquad (26)$$

in which,

$$u' = \frac{l}{E A'_m \delta_{HL}}$$

Graphical Determination of V for Vertical Loads.—Equation (20) gives for V_L:

$$V_L = P \frac{\int_A^R x' \, x \, d\, w}{\int_L^R x^2 \, d\, w} = P \frac{\sum_A^R x' \, x \, \Delta \, w}{\sum_L^R x^2 \, \Delta \, w}$$

The rib is to be considered (Fig. 11) as a cantilever fixed at the right end with the vertical reaction, V_L, on a line passing through the elastic center of gravity of the rib and connected by a rigid arm with the left end. Then, if V_L has a value of unity, the moment at any point in the rib will be $M = V_L \, x = x$, and the vertical displacement of L' due to $V_L = 1$ is:

$$\Delta \, y = \delta_{VL} = \int_L^R M \, x \, d \, w = \int_L^R x^2 \, d \, w$$

This is the denominator of the equation for V_L and is obtained graphically by drawing the moment diagram for the rib with each section, $\Delta \, s$, loaded with its value of $x \, \Delta \, w$ considered as acting vertically at the center of the section.

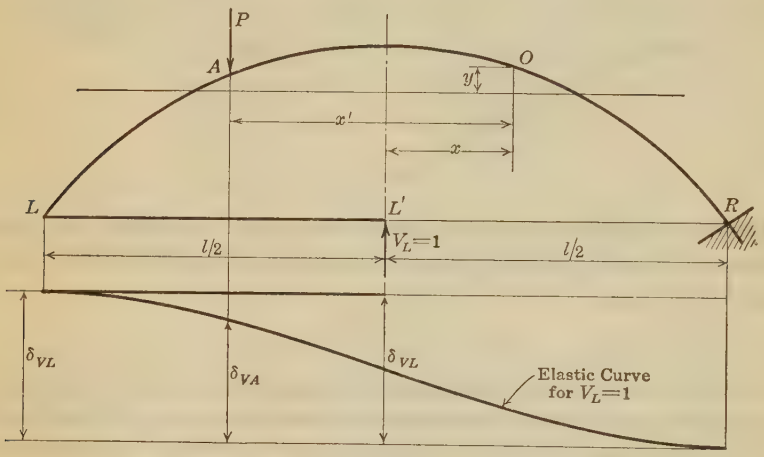


FIG. 11.

If the rib as a cantilever is subjected to a single unit vertical force, P, at any point, A, the moment will be $M = -P \, x' = -x'$ and the vertical displacement of L' due to $P = 1$ will be:

$$\Delta \, y = \int_A^R M \, x \, d \, w = -\int_A^R x' \, x \, d \, w$$

This term is the same as the numerator in the equation for V_L. From Maxwell's theorem, the vertical displacement of L' produced by a unit vertical load at A is equal to the vertical displacement at A produced by a unit vertical V_L at L'. Therefore:

$$\int_A^R x' \, x \, d \, w = \delta_{VA}$$

The value of δ_{VA} is obtained by constructing the moment curve for the rib loaded with the values of $x \Delta w$ and is equal to the ordinate between this curve and the tangent to it at its right end. This curve has already been drawn to determine the value of δ_{VL}. Then, it follows that:

$$V_L = P \frac{\delta_{VA}}{\delta_{VL}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Graphical Determination of M′ for Vertical Loads.—The equation for M' is:

$$M' = P \frac{\int_A^R x' \, d\, w}{\int_L^R d\, w} = P \frac{\sum_A^R x' \, \Delta\, w}{\sum_L^R \Delta\, w}$$

When the rib is divided into a number of sections, either with Δw or Δx a constant, the value of $\sum_L^R \Delta w$ can be computed by direct summation. It is also equal to the angle, θ_{ML}, in Fig. 12; since, when $M = 1$,

$$\theta_{ML} = \int_L^R M \, d\, w = \int_L^R d\, w$$

If a unit value of P is placed at any point, A, of the rib in Fig. 12, the moment at any other point, O, is $M = -P\,x' = -x'$. Then the angular deflection of the tangent at L with respect to the tangent at R will be:

$$\Delta\, \phi = \int_A^R M \, d\, w = -\int_A^R x' \, d\, w$$

This is the numerator in the equation for M'. From Maxwell's theorem, the angular displacement at L produced by a unit vertical load at A is equal to the vertical displacement at A produced by a unit moment at L; hence:

$$\int_A^R x' \, d\, w = \delta_{MA}$$

Under the assumption that M' is unity, the moment at all points on the rib is $M = 1$ and the value of δ_{MA} can be obtained graphically by constructing the bending-moment diagram for the rib with each section loaded with its value of Δw considered as a force acting vertically. The value of δ_{MA} is then the ordinate between this line and the tangent to it at the right end. Then,

$$M' = P \frac{\delta_{MA}}{\sum_L^R \Delta\, w} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Effect of Horizontal Loads.—When the load, P, is inclined or horizontal, it should be resolved into its vertical and horizontal components and the effect of each determined separately. The effect of horizontal forces may be found in a manner similar to that used for vertical loads. If y'' be used to designate the vertical distance from any point, O, on the rib axis to the line of action of the horizontal load, the moment produced by the load becomes $-Py''$

instead of $-Px'$. The formulas used for vertical loading can be applied to horizontal loading by changing x' to y''. Then,

$$\left.\begin{aligned} H_L &= P\,\frac{-\int_A^R y''\,y\,d\,w}{\int_L^R y^2\,d\,w} = P\,\frac{-\sum_A^R y''\,y\,\Delta\,w}{\sum_L^R y^2\,\Delta\,w} \\ V_L &= P\,\frac{\int_A^R y''\,x\,d\,w}{\int_L^R x^2\,d\,w} = P\,\frac{\sum_A^R y''\,x\,\Delta\,w}{\sum_L^R x^2\,\Delta\,w} \\ M' &= P\,\frac{\int_A^R y''\,d\,w}{\int_L^R d\,w} = P\,\frac{\sum_A^R y''\,\Delta\,w}{\sum_L^R \Delta\,w} \end{aligned}\right\} \quad \ldots\ldots(29)$$

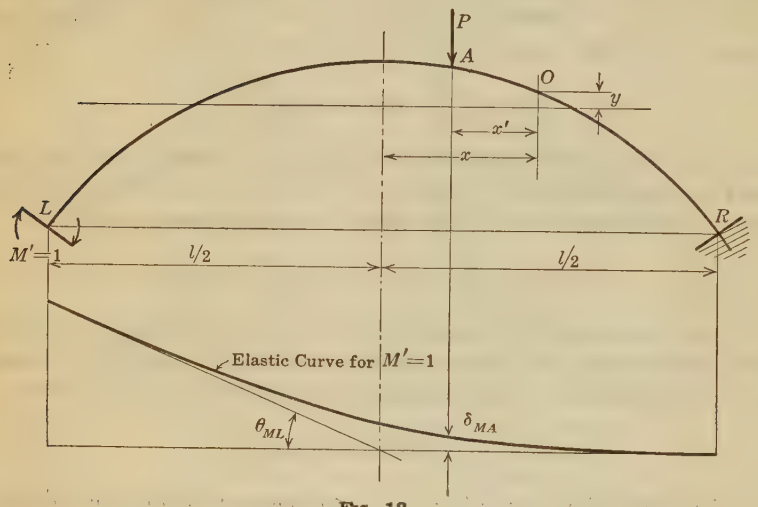


FIG. 12.

Graphical Determination of H for Horizontal Loads.—

$$H_L = P\,\frac{-\sum_A^R y''\,y\,\Delta\,w}{\sum_L^R y^2\,\Delta\,w}$$

The denominator in this equation is the same as that for vertical loads and is found in the same way; it is δ_{HL}.

The numerator is also found as previously. When $P = 1$, the moment at any point, O, is $M = -P\,y'' = -y''$. The horizontal displacement of L' due to $P = 1$ is:

$$\Delta\,x = \int_A^R M\,y\,d\,w = -\int_A^R y''\,y\,d\,w$$

By Maxwell's theorem, this is equal to the horizontal displacement of A produced by a unit horizontal thrust, H, at L' which equals δ'_{HA} (Fig. 13). This is obtained from the elastic curve for the condition, $H = 1$, which is the moment curve for the rib loaded with the values of $y \, \Delta w$ acting horizontally at the centers of divisions. Then,

$$H_L = - P \frac{\delta'_{HA}}{\delta_{HL}} \qquad (30)$$

The negative sign is used because H_L will be opposite in direction to P. Evidently, the value given by Equation (29) is negative.

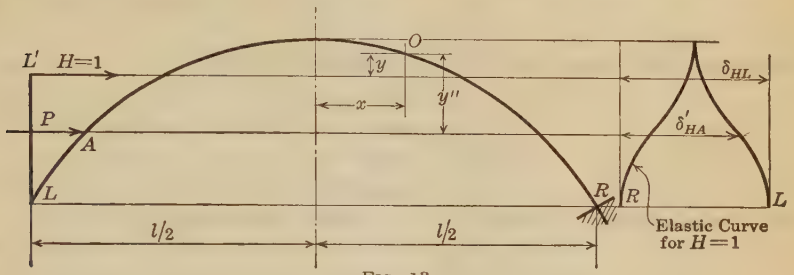


FIG. 13.

Graphical Determination of V_L for Horizontal Loads.—

$$V_L = P \frac{\sum_A^R y'' \, x \, \Delta w}{\sum_L^R x^2 \, \Delta w}$$

The denominator is the same as that for a vertical load and is δ_{VL} which is obtained as explained previously.

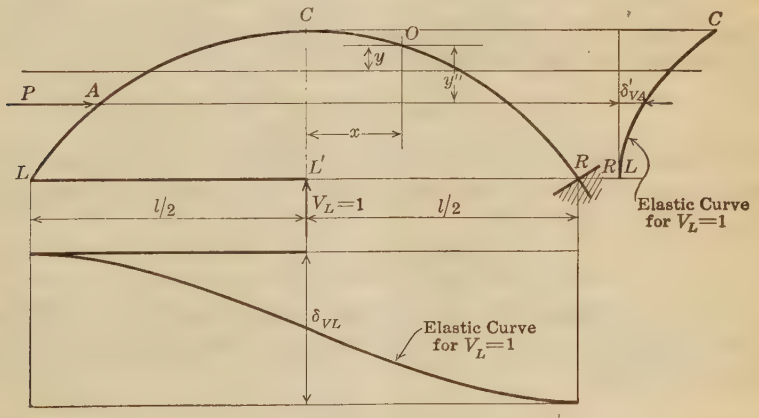


FIG. 14.

With a unit horizontal force, P, acting on the rib, the moment at any point due to P is $M = -Py'' = -y''$ and the vertical displacement of the point of application of V_L produced by P will be:

$$\Delta y = \int_A^R M \, x \, d w = - \int_A^R y'' \, x \, d w$$

This is the numerator of the equation for V_L. By Maxwell's theorem, the vertical displacement of L' produced by a unit horizontal load at A is equal to the horizontal displacement of A due to a unit vertical load at L'. Therefore:

$$\int_A^R y'' \, x \, d\, w = \delta'_{VA}$$

This value is obtained from the elastic curve secured by constructing the bending moment curve for the rib loaded with the values of $x \, \Delta \, w$ acting horizontally at the centers of their respective divisions. V_L is negative when P acts toward the right. Then,

$$V_L = - P \frac{\delta'_{VA}}{\delta_{VL}} \qquad (31)$$

Graphical Determination of M' for Horizontal Loads.—

$$M' = P \frac{\sum_A^R y'' \, \Delta \, w}{\sum_L^R \Delta \, w}$$

The denominator is to be obtained by direct summation as before.

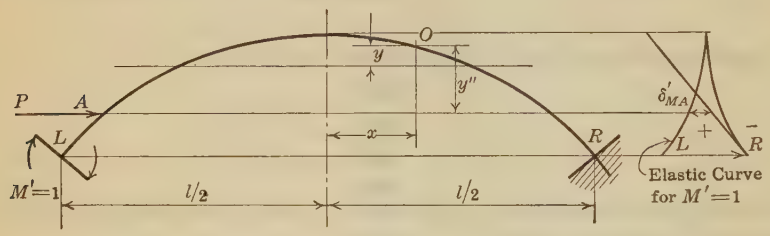


FIG. 15.

The moment of the horizontal force, $P = 1$, is $M = - P \, y'' = - y''$, and the angular displacement of L due to $P = 1$ is:

$$\Delta \, \phi = \int_A^R M \, d \, w = - \int_A^R y'' \, d \, w$$

This is the numerator of the formula for M', but it is also equal by Maxwell's theorem to the horizontal displacement of the point, A, produced by a unit value of M'. Hence,

$$\int_A^R y'' \, d \, w = \delta'_{MA}$$

This value is obtained by drawing the moment curve for the rib loaded with the values of $\Delta \, w$ (since $M = M' = 1$) acting horizontally at the centers of divisions. Then,

$$M' = P \frac{\delta'_{MA}}{\sum_L^R \Delta \, w} \qquad (32)$$

It should be noted that since the rib is considered as a cantilever fixed at the right end, the δ-deformations are always measured as the ordinates to the elastic curve from the tangent to the curve at the right end.

Construction of Influence Lines.—The foregoing graphical constructions give the influence lines for the values of H, V_L, and M'. The influence lines for the moment at the left abutment, M_L, is obtained by combining the values of H, V_L, and M', in accordance with Equation (17) :

$$M_L = M' + H\,b - \frac{V_L\,l}{2}$$

When the reactions at the abutment have been determined, the moments, shears, and thrusts at any other section can easily be computed.

The moment at any point, O, in the rib due to the reactions, is $M' - H_L\,y + V_L\,x$, and the total moment is:

$$M_O = M' + V_L\,x - H_L\,y - M_p \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

in which, M_p is the moment produced by the superimposed loads when the rib is acting as a cantilever with the right end fixed. Thrusts and shears are obtained by ordinary statical methods.

It is usually best for bridge arches to construct the influence lines for unit loading and then to multiply the ordinates by the loads after the critical location of the loads has been determined. The actual load concentrations can be used, or the equivalent uniformly distributed load can be computed from the influence lines. This equivalent uniform load varies with the span, rise, width, and depth of fill over the rib, and should be computed for each section that is being analyzed.

III.—Examples of Graphical Analysis

Deflection of Arch Rib.—When it is desired to determine the deflection of an arch ring under any loading, the elastic line can easily be constructed as shown in Fig. 16.

In this case, three panel points were loaded and the resulting reactions determined. The bending-moment diagram was drawn as shown and then the elastic line was obtained by drawing another bending-moment diagram for the rib, assuming the divisions to be loaded with their respective values of $\frac{M\,\Delta\,s}{E\,I}$. In this case, the rib was divided into twenty sections of such length that all the values of $\frac{\Delta\,s}{E\,I}$ were the same and equal to the constant, C. The bending moments were laid off on the load line taking account of their sign, an arbitrary pole length, p, was chosen, and the funicular polygon, $A' - B'$, was drawn with its sides parallel to the rays of the force polygon. This funicular polygon is the bending-moment curve of the rib loaded with its actual bending moments. The ordinate, d, under any point gives the vertical deflection of that point of the rib.

The deflection, δ, $= d\,p\,C$, in which, $C = \frac{\Delta\,s}{E\,I}$ and p is the pole length measured to the same scale as the load line of the force polygon. This construction could have been made with the span divided into equal $\Delta\,x$ divisions. It would then have been necessary to lay off the values of $\frac{M\,\Delta\,s}{E\,I}$ on the force

diagram instead of the values of M, and the deflection would then have been $\delta = d\,p$.

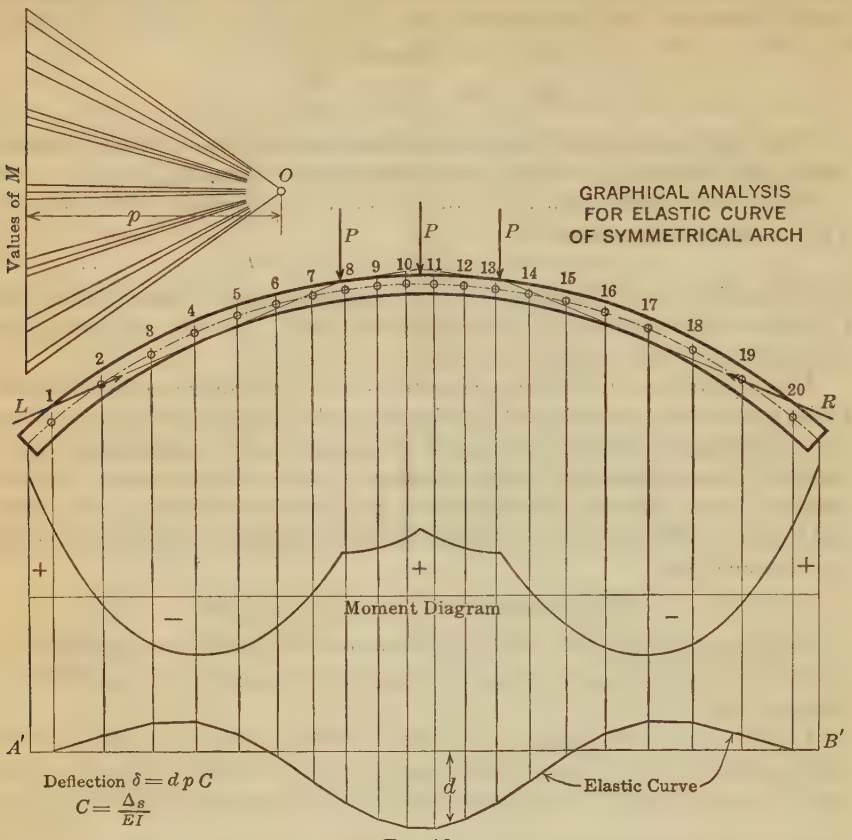


FIG. 16.

If a complete investigation of the deflection of all parts of the rib is desired, it can be made by drawing separately the elastic curve of the rib with a unit load at each panel in succession. From Maxwell's theorem, these elastic curves are also the influence lines for the deflection at the point which was loaded in each case. These influence lines will show the positions of the loading that will produce the maximum deflection and the magnitude of the deflection in the same way as the bending-moment influence lines give the corresponding information regarding the bending moments.

Example of Graphical Analysis of Arch.—A complete graphical determination of H_L, V_L, and M', for unit vertical and horizontal loads is shown in Fig. 17.

The arch rib should be divided into about twenty sections so that all values of either $\frac{\Delta s}{E\,I}$ or Δx are the same. If the values of Δx are made the same, the work of dividing the arch is simplified, but more numerical work is required.

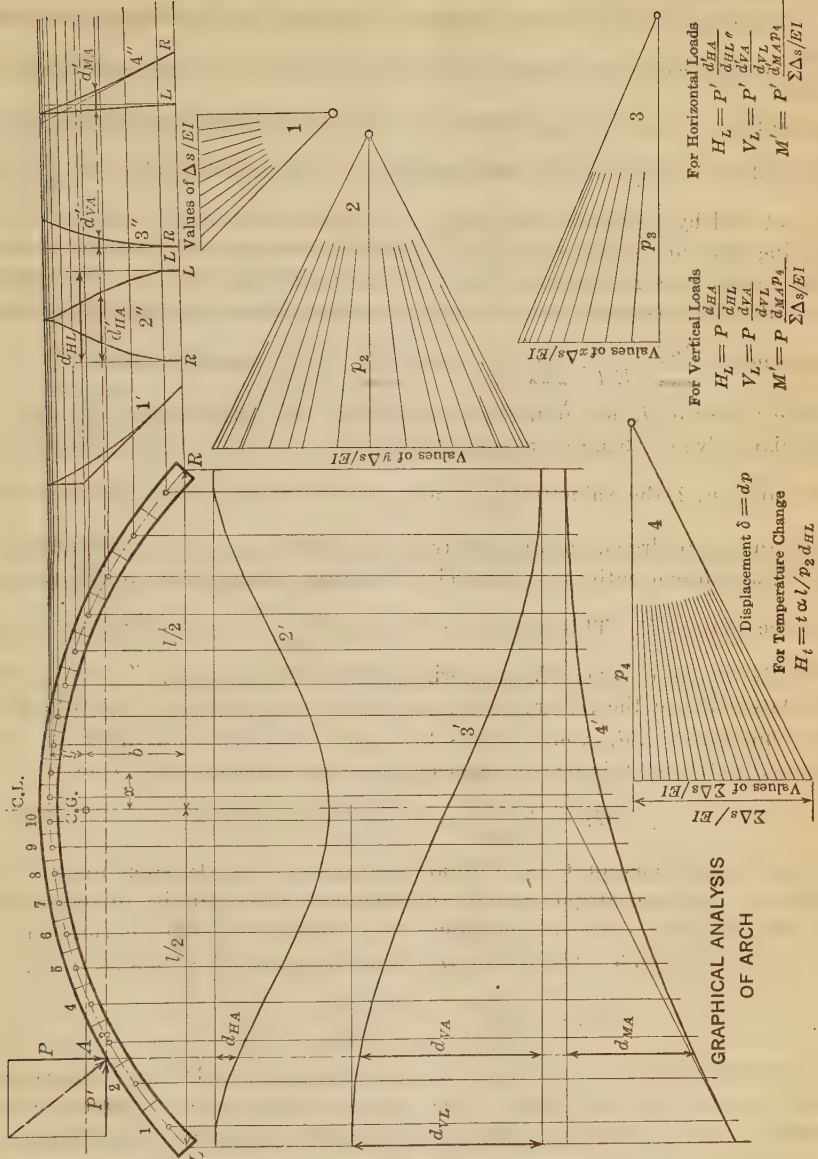

GRAPHICAL ANALYSIS OF ARCH
For Vertical Loads
$H_L = P\ d_{HA}/d_{HL}$
$V_L = P\ d_{VA}/d_{VL}$
$M' = P\ d_{MA}P_4/\Sigma\Delta s/EI$
For Horizontal Loads
$H_L = P'\ d_{HA}/d_{HL}$
$V_L = P'\ d_{VA}/d_{VL}$
$M' = P'\ d_{MA}P_4/\Sigma\Delta s/EI$
For Temperature Change
$H_t = t\,\alpha\,l/p_2\,d_{HL}$
Displacement $\delta = dp$
Values of $\Delta s/EI$
Values of $y\,\Delta s/EI$
Values of $x\,\Delta s/EI$
Values of $\Sigma\Delta s/EI$
$\Sigma\Delta s/EI$
C.L.
C.G.
$l/2$
$l/2$
d_{HA}
d_{VA}
d_{MA}
d_{VL}
d_{HL}
d'_{VA}
d'_{HA}
d'_{MA}
P
P'
L
R

Fig. 17.

After the arch divisions have been established it is necessary to determine the location of the gravity axis from which the values of y and x are to be measured. Since the rib is symmetrical, the vertical axis must be the center line of the arch.

The horizontal axis is found graphically (Fig. 17) by considering the values of $\frac{\Delta s}{EI}$ to be forces acting horizontally at the centers of divisions and determining the line of action of the resultant by the usual method for parallel forces. This is done by means of the force polygon, 1, in which the values of $\frac{\Delta s}{EI}$ are laid out and by the funicular polygon, 1, having sides parallel to the corresponding rays of the force polygon, 1. The intersection of the sides parallel to the outside rays gives the location of the horizontal gravity axis. The values of x and y are measured from the vertical and horizontal axes thus established. The values of $\sum \frac{x \Delta s}{EI}$, $\sum \frac{y \Delta s}{EI}$, and $\sum \frac{x y \Delta s}{EI}$, should be zero.

The values of H are found by constructing the force polygon, 2, and the funicular polygon "bending-moment diagrams", 2′ and 2″. To construct the force polygon, 2, the values of $\frac{y \Delta s}{EI}$ are laid off algebraically on the load line and the rays are drawn with an arbitrary pole length, p_2. The elastic lines, 2′ and 2″, are drawn with sides parallel and normal, respectively, to the rays of the force polygon, 2. This is because the forces, $\frac{y \Delta s}{EI}$ are considered as acting vertically for the polygon, 2′, and horizontally for the polygon, 2″. The actual δ-displacements of the rib are then equal to the pole length, p_2, multiplied by the ordinates, d_{HA}, d_{HL}, etc., but as the same pole length was used for both funicular polygons, this term disappears from the expression for H, or,

$$H_L = P \frac{\delta_{HA}}{\delta_{HL}} = P \frac{p_2 d_{HA}}{p_2 d_{HL}} = P \frac{d_{HA}}{d_{HL}}$$

The constructions for V_L and M' are similar to the construction for H. To obtain V_L, the funicular polygon, 3′, is drawn with sides parallel to the rays of the force polygon, 3, and the polygon, 3″, is drawn with sides normal to the rays of the polygon, 3. The pole length cancels out as before, and,

$$V_L = P \frac{\delta_{VA}}{\delta_{VL}} = P \frac{p_3 d_{VA}}{p_3 d_{VL}} = P \frac{d_{VA}}{d_{VL}}$$

The force polygon, 4, and the funicular polygons, 4′ and 4″, are drawn in a similar manner with the sides of the polygon, 4′, parallel and those of the polygon, 4″, perpendicular to the rays of the polygon, 4. The pole length, p_4, does not cancel out of the expression for M' as it appears only in the numerator:

$$M' = P \frac{\delta_{MA}}{\sum \frac{\Delta s}{EI}} = P \frac{p_4 d_{MA}}{\sum \frac{\Delta s}{EI}}$$

It should be noted that if the division of the ring is such that $\frac{\Delta s}{E I}$ is constant, this value of $\frac{\Delta s}{E I} = C$ need not be considered in the construction of the force polygons, merely the values of y and x being laid off instead of $\frac{y \Delta s}{E I}$ and $\frac{x \Delta s}{E I}$. Also, if n is the number of divisions, the load line for the force polygon, 4, becomes merely n equal divisions and the pole length, p_4, is measured with the length of one division as a unit. The expressions for H and V_L are as before, but M' becomes:

$$M' = P \frac{p_4 \, d_{MA}}{n}$$

As regards bending, the value of $\frac{\Delta s}{E I} = C$ need not be considered in the determination of the reactions for live load, as these depend only on the relative deformations, but for rib-shortening, temperature change, and movements of the abutments, the value, C, must be used. When the load lines are laid out with values of y and x only, the absolute displacements are given by,

$$\delta = p \cdot \frac{\Delta s}{E I} \cdot d$$

in which d is the proper ordinate measured from the funicular polygon, and p is the corresponding pole length.

The thrust due to temperature change is (Equation (25)):

$$H_t = \frac{t \alpha l}{\delta_{HL}} = \frac{t \alpha l}{p_2 \, d_{HL}}$$

The rib-shortening thrust is (Equation (26)):

$$H_{RS} = \frac{H l}{E A'_m \delta_{HL}} = \frac{H l}{E A'_m p_2 \, d_{HL}} .$$

The value of A'_m is to be obtained numerically by taking the reciprocal of the average value of $\frac{1}{A \cos \phi}$ for all sections. For the computation of A'_m, the rib must be divided into equal Δx divisions.

Analysis of Arch with Fixed Loads.—Although it is generally desirable to determine influence lines that give the effect of single loads in any position, especially for bridge arches which carry moving loads, it may be simpler in some cases to make a direct analysis for certain conditions of loading, such as wind load on the side of a building arch, or full live load where moving loads do not have to be considered. The method just explained also applies, of course, to tall symmetrical arches for buildings such as are shown in Fig. 18, for which influence lines can be used to determine the effects of full horizontal and vertical loads.

If, in the derivation of Equations (20), the moment, M_p, produced by the superimposed loads is left in the numerator without the substitution of the

values for single concentrated loads, considering the abutments rigid, the formulas become:

$$\left.\begin{aligned} H &= \frac{\int_L^R M_p\, y\, d\, w}{\int_L^R y^2\, d\, w + \int_L^R \frac{\cos\,\phi\, d\, x}{E\, A}} \\ V_L &= \frac{\int_L^R M_p\, x\, d\, w}{\int_L^R x^2\, d\, w} \\ M' &= \frac{\int_L^R M_p\, d\, w}{\int_L^R d\, w} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (34)$$

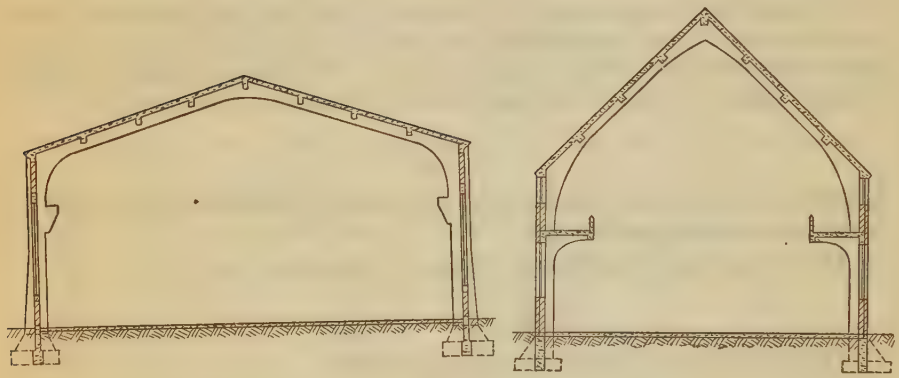


FIG. 18.

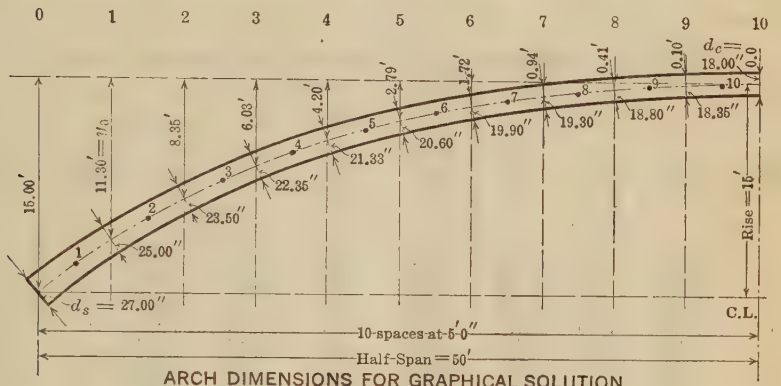
The denominators of these equations are the values of δ_{HL}, δ_{VL}, and $\sum \frac{\Delta\, s}{E\, I}$, which appear in Equations (24), (27), and (28); their determination has already been explained. The numerators, $\int M_p\, y\, d\, w$, $\int M_p\, x\, d\, w$, and $\int M_p\, d\, w$, are, respectively, the horizontal, vertical, and angular displacements of the left end of the rib produced by the loads when the rib is a cantilever fixed at the right-hand end. These displacements can easily be found by laying off the values of $\frac{M_p\, \Delta\, s}{E\, I}$ in a force diagram and constructing the two funicular polygons, considering them as forces acting horizontally to obtain the x-displacement and vertically to get the y-displacement. The angular displacement, $\sum \frac{M_p\, \Delta\, s}{E\, I}$, can be obtained by direct summation and is the length of the load line used previously.

The application of this method to the analysis of a very large reinforced concrete arched conduit section with monolithic invert forming a closed ring

has been fully explained by the writer.* It is believed to be the simplest possible method for solving such a problem. The summations can be made either analytically or graphically.

Numerical Example of Graphical Solution.—An application of the graphical method for determining the values of H, V, M_c, and M_L, for unit loads will be made for the rib shown in Fig. 19. It is assumed that the arch has an earth fill and that a slice of the arch only 1 ft. in width is considered. For this arch rib, the dimensions are:

$l = 100$ ft. $\qquad d_c = 18$ in.
$r = 15$ ft. $\qquad d_s = 27$ in.



ARCH DIMENSIONS FOR GRAPHICAL SOLUTION

FIG. 19.

The reinforcement at the crown will consist of ⅞-in. round rods spaced 15 in. on centers, 2½ in. from each face of the rib, and at the springing, ⅞-in. round rods spaced 5 in. on centers, 2½ in. from each face.

For the purpose of analysis, the rib will be divided into twenty divisions with Δx equal to 5 ft. in all cases. The computation of the moment of inertia at each division point is given in Table 1.

TABLE 1.—ELEMENTS OF SECTIONS OF CONCRETE ARCH RIB.

Point.	d, in inches.	I (concrete).	A_s, in square inches.	$d - 5$ inches	$14 \times I$ (steel).	Total I, in inches^4.
Spring 0	27.00	19 670	2.884	22.0	4 890	24 560
1	25.00	15 625	2.884	20.0	4 040	19 665
2	23.50	12 980	2.884	18.5	2 455	15 435
3	22.35	11 170	1.924	17.4	2 028	13 198
4	21.33	9 700	1.924	16.3	1 796	11 496
5	20.60	8 730	1.924	15.6	1 640	10 370
6	19.90	7 880	0.962	14.9	746	8 626
7	19.30	7 180	0.962	14.3	687	7 867
8	18.80	6 630	0.962	13.8	641	7 271
9	18.35	6 170	0.962	13.4	600	6 770
Crown 10	18.00	5 830	0.962	13.0	569	6 399

These computations were made for the most part with a 10-in. slide-rule, with corresponding accuracy. The rib was laid out to a scale of ¼ in. to 1 ft.

* *Engineering Record*, October 16, 1915, p. 486.

For the computation of the values of $\frac{\Delta s}{E I}$, $\frac{x \Delta s}{E I}$, and $\frac{y \Delta s}{E I}$, for each division, average values of I, x, and y, were used. These values are given in Table 2. The modulus of elasticity of the concrete, E, is taken as 2 000 000 lb. per sq. in. and n as 15.

The location of the gravity axis along which H acts is computed from the summations of Δw and $y_0 \Delta w$ and is:

$$y_c = \frac{\Sigma y_0 \Delta w}{\Sigma \Delta w} = \frac{8.161}{2.601} = 3.14 \text{ ft.}$$

The values of y are then computed from this axis and are equal to $y = 3.14 - y_0$.

TABLE 2.—Dimensions and Factors for Concrete Arch.

Division number.	I, in inches^4	Δs, in feet.	$\frac{\Delta s}{E I}\frac{12}{10^9}$	y_0, in feet.	$\frac{y_0 \Delta s}{E I}\frac{144}{10^9}$	y, in feet.	$\frac{y \Delta s}{E I}\frac{144}{10^9}$	x, in feet.	$\frac{x \Delta s}{E I}\frac{144}{10^9}$
1	22 112	6.250	0.1413	13.15	1.858	−10.01	−1.415	47.5	6.71
2	17 550	5.820	0.1664	9.825	1.635	— 6.685	−1.112	42.5	7.07
3	14 316	5.530	0.1930	7.190	1.387	— 4.050	−0.782	37.5	7.24
4	12 347	5.335	0.2160	5.110	1.107	— 1.970	−0.426	32.5	7 02
5	10 933	5.215	0.2384	3.495	0.834	— 0.355	−0.086	27.5	6.56
6	9 498	5.130	0.2700	2.255	0.609	0.885	0.238	22.5	6.08
7	8 246	5.065	0.3070	1.331	0.409	1.809	0.554	17.5	5.37
8	7 569	5.027	0.3320	0.676	0.224	2.464	0.818	12.5	4.15
9	7 020	5.010	0.3572	0.256	0.092	2.884	1.029	7.5	2.68
10	6 584	5.000	0.3800	0.025	0.009	3.115	1.182	2.5	0.95
Summation.....	.....	.....	2.601	.....	.....	.....	0.000	.....	53.85

The horizontal gravity axis is located by constructing the force polygon, 1, and the funicular polygon, 1 (Fig. 20). This serves as a check on the computation.

The horizontal thrust is obtained by drawing the funicular polygons, 2 and 2′, with sides parallel and perpendicular, respectively, to the rays of the force polygon, 2. H for a unit load is $\frac{d_{HA}}{d_{HL}}$, the values for which as scaled from Fig. 20 are given in Table 3.

The values of the vertical reaction at the left end are obtained from the funicular polygon, 3, which has its sides parallel to the rays of the force polygon, 3. For a unit load, $V_L = \frac{d_{VA}}{d_{VL}}$

The moment, M', is taken from the funicular polygon, 4, which has sides parallel to the rays of the force polygon, 4. For a unit loading:

$$M' = \frac{p_4 d_{MA}}{\Sigma \Delta w} = 0.961 \, d_{MA}$$

since,

$$\Sigma \Delta w = 2.601 \times 2 \times \frac{144}{10^9} = \frac{749}{10^9}$$

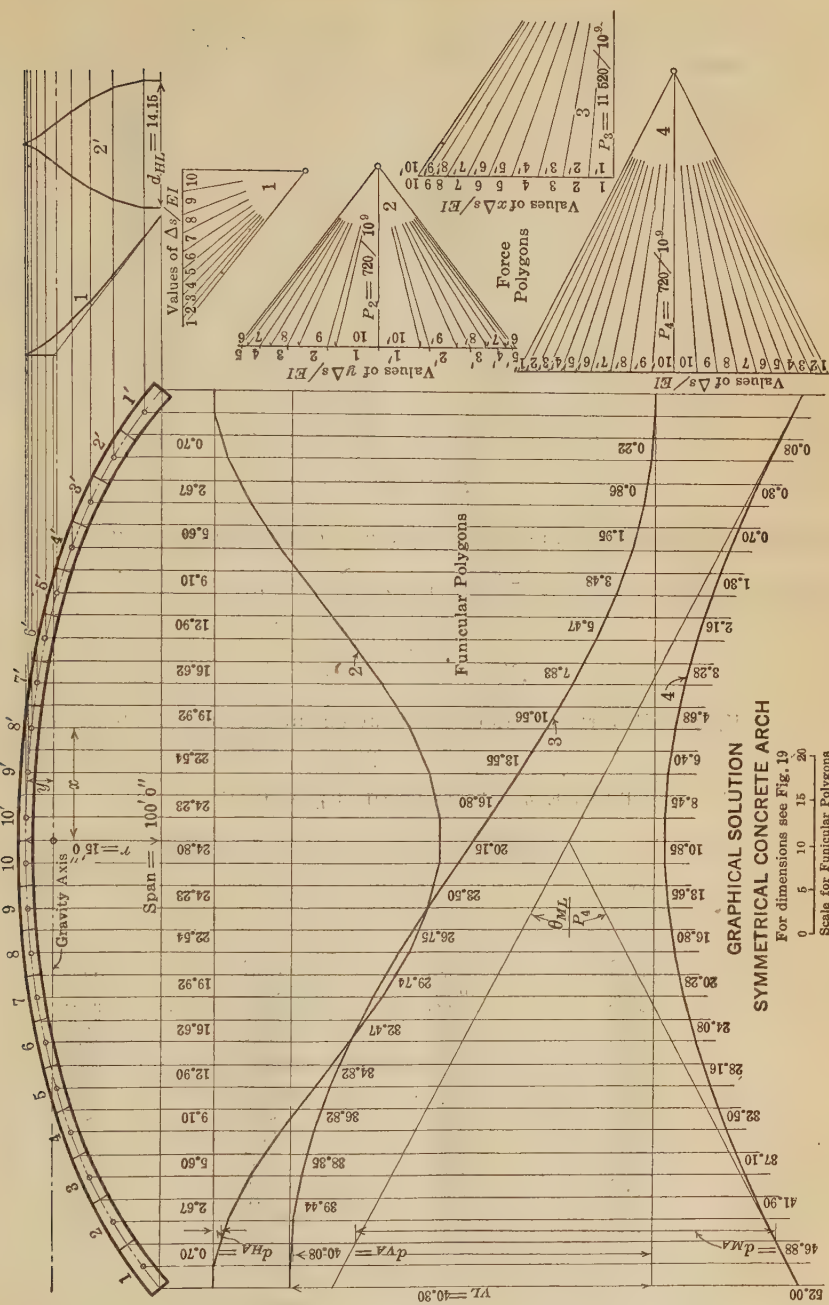

GRAPHICAL SOLUTION
SYMMETRICAL CONCRETE ARCH
For dimensions see Fig. 19
Scale for Funicular Polygons
Funicular Polygons
Force Polygons
Values of $\Delta s/EI$
Values of $y\Delta s/EI$
Values of $x\Delta s/EI$
Values of $\Delta y/EI$
$P_1 = 720/10^9$
$P_2 = 720/10^9$
$P_3 = 11\,520/10^9$
$P_4 = 720/10^9$
$d_{HL} = 14.15$
Span = 100′ 0″
$r = 15.0$
Gravity Axis
$d_{HA} = 0.70$
$d_{VA} = 40.08$
$V_L = 40.30$
$d_{MA} = 46.88$
52.00

FIG. 20.

TABLE 3.—Horizontal Thrust.

Point.	d_{HA}.	$H = \frac{d_{HA}}{d_{HL}}$.
1	0.70	0.049
2	2.67	0.189
3	5.60	0.396
4	9.10	0.643
5	12.90	0.912
6	16.62	1.174
7	19.92	1.407
8	22.54	1.592
9	24.23	1.712
10	24.80	1.752
d_{HL} =	14.15	.....

and,

$$p_4 = \frac{720}{10^9}$$

The slope of the angle, θ_{ML}, as shown in Fig. 20, can also be used instead of $\frac{p_4}{\Sigma \Delta w}$, and, in this case, it is $\frac{52.00}{50} = 1.04 = \frac{\Sigma \Delta w}{p_4}$, or,

$$M' = \frac{\delta_{MA}}{\theta_{ML}} = \frac{d_{MA}}{1.04}$$

Values of V_L and M' for the various points along the arch are given in Table 4.

TABLE 4.—Values of V_L and M'.

Point.	d_{VA}.	$V_L = \frac{d_{VA}}{d_{VL}}$.	d_{MA}.	$M' = \frac{d_{MA}}{\theta_{ML}}$.
1	40.08	0.994	46.88	45.10
2	39.44	0.979	41.90	40.25
3	38.35	0.952	37.10	35.66
4	36.82	0.914	32.50	31.22
5	34.82	0.864	28.16	27.05
6	32.47	0.806	24.08	23.15
7	29.74	0.738	20.28	19.50
8	26.75	0.664	16.80	16.14
9	23.50	0.583	13.65	13.12
10	20.15	0.500	10.85	10.43
9′	16.80	0.417	8.45	8.12
8′	13.55	0.336	6.40	6.15
7′	10.56	0.262	4.68	4.50
6′	7.83	0.194	3.28	3.15
5′	5.47	0.136	2.16	2.08
4′	3.48	0.086	1.30	1.25
3′	1.95	0.048	0.70	0.67
2′	0.86	0.021	0.30	0.29
1′	0.22	0.006	0.08	0.08
d_{VL} =	40.30	.....	θ_{ML} = 1.04	.....

The moment at the springing is given by $M_L = M' + H\,b - \frac{V\,l}{2}$ and at the crown, by $M_c = M' - H\,y_c - P\,x'$. $P\,x'$ is the moment of the load about the center. The value of b is $r - y_c = 11.86$ ft. The values of M_L and M_c for unit loads are given in Table 5.

TABLE 5.—Moment at the Springing Line and the Crown.

Load at point.	M'.	$H\,y_c$.	$P\,x'$.	M_c, in foot-pounds.	$H\,b$.	50 V_L.	M_L, in foot-pounds.
1	45.10	0.15	45.0	−0.05	0.58	49.70	−4.02
2	40.25	0.59	40.0	−0.34	2.24	48.95	−6.46
3	35.66	1.24	35.0	−0.58	4.70	47.60	−7.24
4	31.22	2.02	30.0	−0.80	7.62	45.70	−6.86
5	27.05	2.86	25.0	−0.81	10.81	43.20	−5.84
6	23.15	3.69	20.0	−0.54	13.93	40.30	−3.22
7	19.50	4.42	15.0	+0.08	16.68	36.90	−0.72
8	16.14	5.00	10.0	+1.14	18.88	33.20	+1.82
9	13.12	5.37	5.0	+2.75	20.32	29.15	+4.29
10	10.43	5.50	....	+4.93	20.78	25.00	+6.21
9′	8.12	5.37	....	+2.75	20.32	20.85	+7.59
8′	6.15	5.00	....	+1.15	18.88	16.80	+8.23
7′	4.50	4.42	....	+0.08	16.68	13.10	+8.08
6′	3.15	3.69	....	−0.54	13.93	9.70	+7.38
5′	2.08	2.86	....	−0.78	10.81	6.80	+6.09
4′	1.25	2.02	....	−0.77	7.62	4.30	+4.57
3′	0.67	1.24	....	−0.57	4.70	2.40	+2.97
2′	0.29	0.59	....	−0.30	2.24	1.05	+1.48
1′	0.08	0.15	....	−0.07	0.58	0.30	+0.36

These computed values of M_c and M_L should be plotted as a check so that any values which do not lie on a smooth curve may be corrected.

The temperature thrust is given by Equation (25) as:

$$H_t = \frac{t\,\alpha\,l}{\delta_{HL}}$$

The value of δ_{HL} is equal to $p_2\,d_{HL}$ and, hence,

$$H_t = \frac{t\,\alpha\,l}{p_2\,d_{HL}} = \frac{t\,\alpha\,l}{12 \times 14.15 \times \frac{720}{10^9}} = \frac{t\,\alpha\,l}{0.0001223}$$

$$= t\,\frac{0.000006 \times 1200}{0.0001223} = 58.9\,t$$

and the moment at the crown in foot-pounds is:

$$M_t = -3.14 \times 58.9\,t = 185.0\,t$$

The rib-shortening thrust is:

$$H_{RS} = H\,u' = \frac{H\,l}{E\,A'_m\,\delta_{HL}}$$

$$\delta_{HL} = p_2\,d_{HL} = 12 \times 14.15 \times \frac{720}{10^9} = 0.0001223$$

$$E = 2\,000\,000$$

$$l = 1\,200$$

The calculation of the value of A'_m is given herewith. The values of $\frac{1}{A \cos \phi}$ are calculated from the average values of the cos ϕ and the depth of rib for each section of rib. A is in square feet:

Section.	$A \cos \phi$.
1	0.577
2	0.577
3	0.578
4	0.586
5	0.597
6	0.608
7	0.621
8	0.634
9	0.647
10	0.661
Total	6.086
Average	0.6086

$$A'_m = \frac{1}{0.6086} = 1.643 \text{ sq. ft.} = 236.6 \text{ sq. in.}$$

Then,

$$u' = \frac{1\,200}{2\,000\,000 \times 236.6 \times 0.0001223} = 0.0207$$

and,

$$H_{RS} = -0.0207\ H.$$

Care must be taken to see that the formulas are kept homogeneous, with all dimensions expressed in inches or in feet, using the same unit in computing E and I and other constants involving lengths or areas.

IV.—The Arch Axis

In order that the formulas for thrusts and moments which have been derived may be solved by actual integration, it is necessary to write an equation for the shape of the arch axis and for the moment of inertia at any point. Such formulas are easily found for parabolic and circular ribs; but the best shape for the arch axis is neither parabolic nor circular. In order to reduce the bending moments, it is desirable to lay out the axis of the rib so that its curve will follow the pressure line of the loads as closely as possible. This curve is parabolic for a load uniformly distributed over the span. As the intensity of loading toward the abutment increases in comparison with the load at the crown, the pressure line between the crown and springing line rises and the angle of inclination at the springing line increases. There is a considerable difference between the pressure line for a flat, open spandrel arch and for a filled arch with a high rise.

The question also arises as to whether or not the arch axis should be located for the dead load only, or for the dead load plus part of the full live load. The addition of part of the full uniform live load to the dead load has

the effect of making the pressure line more nearly parabolic than it would be for the dead load only. The dead load pressure line, therefore, will lie below the arch axis at the crown and springing line and above it in the haunch so that a negative moment (tending to compress the bottom side of the rib) will be produced at the crown and springing line by the dead load. As the critical moment in the crown is positive and that in the springing line is usually negative, the use of part of the live load for locating the axis has the effect of decreasing the stress at the crown and increasing it at the springing line. It is apparent then that there cannot be much gain in economy by considering the live load in laying out the axis; that method, however, may sometimes be used to good advantage when the rise is high and the effects of temperature variation and rib-shortening are slight. In the latter case the positive moment at the springing line may control and the use of half the live load in locating the axis will have the effect of equalizing the positive and negative moments.

To be of practical value, a formula for the arch axis must give results close to the actual pressure line for the loading on the arch, and must be susceptible to easy mathematical treatment. The formula here derived has been found to comply with both these requirements satisfactorily.

The load per unit length of span will be called w_c at the crown, w_s at the springing line, and w_x at any point which is distant, x, from the center line of the span. Also, $W = \int_0^x w_x \, dx$. It will then be assumed that the increase in the loading, w_x, is proportional to the ordinate, y_0 (Fig. 21). Then,

$$w_x = w_c + \lambda y_0$$

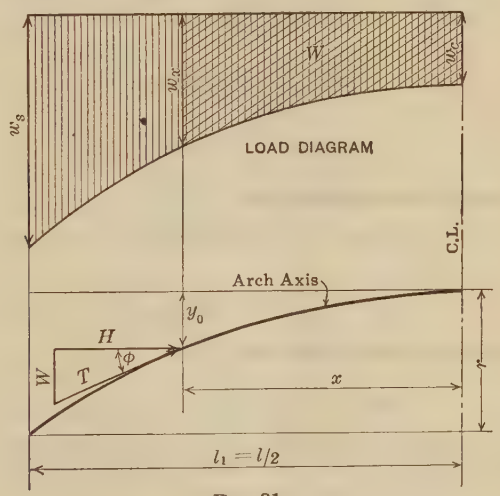


FIG. 21.

At the springing line,

$$w_s = w_c + \lambda r$$

Hence,

$$\lambda = \frac{w_s - w_c}{r}$$

It follows that,

$$w_x = w_c + \frac{y_0}{r}(w_s - w_c) = w_c\left[1 + \frac{y_0}{r}\left(\frac{w_s}{w_c} - 1\right)\right]$$

Let,

$$\frac{w_s}{w_c} = g$$

then,

$$w_x = w_c\left[1 + \frac{y_0}{r}(g - 1)\right] \quad \ldots\ldots(35)$$

At any section,

$$\frac{d\,y_0}{d\,x} = \frac{W}{H}$$

and,

$$\frac{d^2\,y_0}{d\,x^2} = \frac{1}{H}\,\frac{d\,W}{d\,x}$$

As,

$$\frac{d\,W}{d\,x} = w_x, \quad \frac{d^2\,y_0}{d\,x^2} = \frac{w_x}{H} \quad \ldots\ldots(36)$$

Substituting in Equation (35):

$$\frac{d^2\,y_0}{d\,x^2} = \frac{w_c}{H}\left[1 + \frac{y_0}{r}(g - 1)\right]$$

Placing $\frac{x}{l_1} = z$,

$$\frac{d^2\,y_0}{d\,z^2} = \frac{l_1^2\,w_c}{H} + \left[\frac{l_1^2\,w_c}{H} \cdot \frac{y_0}{r}(g - 1)\right]$$

Then if,

$$k^2 = \frac{l_1^2\,w_c}{H\,r}(g - 1)$$

$$\frac{d^2\,y_0}{d\,z^2} = \frac{l_1^2\,w_c}{H} + k^2\,y_0 \quad \ldots\ldots(37)$$

The general solution of this equation is:

$$y_0 = C_1 + C_2 \cosh z\,k$$

or,

$$y_0 = C_2\left(\cosh z\,k + \frac{C_1}{C_2}\right)$$

Equation (35) may be put in the form:

$$y_0 = \frac{r}{g - 1}\left(\frac{w_x}{w_c} - 1\right)$$

Then comparing these two formulas for y_0, it is seen that the equation of the arch axis must be:

$$y_0 = \frac{r}{g - 1}(\cosh z\,k - 1) \quad \ldots\ldots(38)$$

As at the springing line, $y_0 = r$, and $z = 1$, substituting in Equation (38),

$$r = \frac{r}{g - 1}(\cosh k - 1)$$

Hence,

$$\cosh k = g$$

At the crown,

$$y_0 = \frac{r}{g - 1}(1 - 1) = 0$$

Then,

$$\frac{w_s}{w_c} = g = \cosh k$$

or,

$$k = \cosh^{-1} g = \log (g + \sqrt{g^2 - 1}) \quad \ldots\ldots (39)$$

The derivation of the formula for the arch axis may also be checked by differentiation.

From Equation (38):

$$y_0 = \frac{r}{g - 1}(\cosh z\, k - 1)$$

Differentiating,

$$\frac{d\ y_0}{d\ z} = \frac{r\ k}{g - 1} \sinh z\, k$$

$$\frac{d^2\ y_0}{d\ z^2} = \frac{r\ k^2}{g - 1} \cosh z\, k$$

or,

$$\frac{d^2\ y_0}{d\ x^2} = \frac{r\ k^2}{l_1^2\ (g - 1)} \cosh z\, k$$

From Equation (37):

$$k^2 = \frac{l_1^2\ w_c}{H\ r}(g - 1)$$

or,

$$\frac{r\ k^2}{l_1^2\ (g - 1)} = \frac{w_c}{H}$$

Hence,

$$\frac{d^2\ y_0}{d\ x^2} = \frac{w_c}{H} \cosh z\, k \quad \ldots\ldots (40)$$

Substituting Equation (38) in Equation (35), it follows that:

$$w_x = w_c\ (1 + \cosh\ z\, k - 1) = w_c\ \cosh\ z\, k$$

Combining with Equation (40):

$$\frac{d^2\ y_0}{d\ x^2} = \frac{w_x}{H}$$

The curves given by Equation (38) have been found to correspond closely with the actual pressure line for arch bridges of all types. This formula gives much nearer results than the one derived on the assumption that the load, w_x, varies with the ordinates of a parabola the apex of which is at the center of the span.

As the value of g approaches unity, the curve given by Equation (38) approaches a parabola in shape. For the special case of a uniform load when $g = 1$, this equation has no meaning, the expression for the axis then becomes:

$$y_0 = r\ z^2 \quad \ldots\ldots (38a)$$

which is the equation of a parabola.

As the value of g increases, the curve of the axis becomes higher, and the tangent at the springing line becomes steeper.

In order to designate the shape of the arch axis, it seems advisable to use the ratio, $\frac{y_0}{r}$, of the ordinate at the quarter-point of the span to the total rise. This ratio will be called N. The value of N is 0.25 for the parabolic arch when $g = 1$, decreasing as the load at the springing line becomes proportionately higher. In designing a new arch, the value of g is computed first from w_s and w_c, but in investigating a design already completed, the value of N can best be computed directly from the ordinates of the rib axis. Using the corresponding value of g, the axis given by Equation (38) should be close to the actual axis if that has been made to follow the pressure line for the actual loading.

Equation (38) is derived for loads that are not concentrated, but it can be applied to open spandrel arches with columns, by considering the panel loads as distributed over the panel lengths. The value of w_s, the load at the springing line, in that case is not to be calculated with the actual load on the column at that point, but with the load which would be there if the typical spandrel construction on the span were continued another panel beyond. This last load does not actually come on the arch, but must be used in computing the value of g. Assuming the panel lengths to be equal, the end panel load would be taken the same as the panel load next to the end, with only the difference made by the longer column and the greater thickness of rib. This panel load is then divided by the full panel length as if the arch rib were continued back into the abutment.

When panel loads are concentrated on the rib by columns, there will be dead load moments in the rib due to the concentrations, because the pressure curve of the loads will be a broken line whereas the arch axis is a smooth curve. If the axis is properly laid out, these dead load moments should be positive at the panel points and negative between the panel points. The size of this bending will depend on the total number of panels to the span, the variation between pressure line and arch axis being smaller for relatively short panel lengths.

When the value of y_0 at the quarter-point is known, the value of g may be computed from the formula derived as follows:

From Equation (38):

$$y_0 = \frac{r}{g - 1}(\cosh z\, k - 1)$$

For the quarter-point, $z = \frac{1}{2}$, therefore,

$$y_{\frac{1}{4}} = \frac{r}{g - 1}\left(\cosh \frac{1}{2} k - 1\right)$$

but,

$$\cosh \frac{1}{2} k = \sqrt{\frac{\cosh k + 1}{2}} = \sqrt{\frac{g + 1}{2}}$$

then,

$$y_{\frac{1}{4}} = \frac{r}{g-1}\left(\sqrt{\frac{g+1}{2}} - 1\right) = \frac{r}{2+\sqrt{2\,(g+1)}}$$

from which,

$$g = \frac{1}{2}\left(\frac{r}{y_{\frac{1}{4}}} - 2\right)^2 - 1$$

or,

$$g = \frac{1}{2}\left(\frac{1}{N} - 2\right)^2 - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

in which,

$$N = \frac{y_{\frac{1}{4}}}{r}$$

From this equation, the value of g can be computed. In investigating a finished design, when the value of g computed from the actual quarter-point ordinate does not correspond with the g computed from the actual weights, it is best to use an ideal value of w_c equal to $\frac{w_s}{g}$ computed from the actual w_s and the value of g obtained from Equation (41).

Angle of Inclination of the Arch Axis.—The angle between the horizontal and the tangent to the axis at any point will be called ϕ. At the springing line it is ϕ_s and at the crown it is zero.

Equation (38) is:

$$y_0 = \frac{r}{g-1}(\cosh z\,k - 1)$$

As $\tan \phi = \frac{d\,y_0}{d\,x}$, by differentiation is obtained:

$$\tan \phi = \frac{r\,k}{l_1\,(g-1)} \sinh z\,k \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

For the springing line, $z = 1$, and,

$$\tan \phi_s = \frac{r\,k}{l_1\,(g-1)} \sinh k$$

or,

$$\tan \phi_s = \frac{r\,k\,\sqrt{g^2-1}}{l_1\,(g-1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

Similarly, for the quarter-point:

$$\tan \phi_{\frac{1}{4}} = \frac{r\,k}{l_1\,\sqrt{2\,(g-1)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

Tables for Determining Arch Axis.—Table 6 has been computed from Equation (38) and gives the values of $\frac{y_0}{r}$ with twenty subdivisions of the span for values of N varying from 0.25 to 0.15. Values not given in this table can be obtained by interpolation, and the ordinates of the rib are obtained simply

by multiplying by r. Table 6 will be found valuable in laying out the axis of the rib when N or g has been computed.

Table 7 gives the inclination of the tangent to the rib in terms of the intercept on the horizontal through the crown, as shown in Fig. 22.

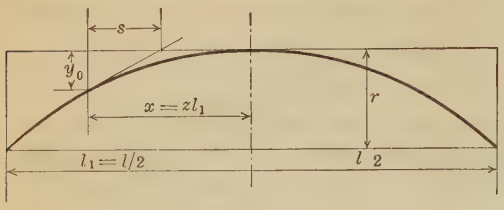


FIG. 22.

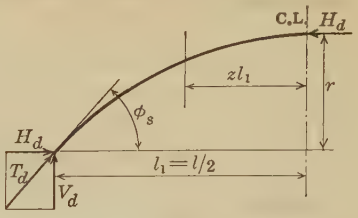


FIG. 23.

These data will prove useful in drawing the curve of the axis determining the tangents at the different points as the points are plotted.

Reactions from Dead Load.—The thrust and the horizontal and vertical reactions for the loading for which the rib axis is laid out can readily be computed. If the rib is laid out for dead load only, there will be no dead load moment in the rib except that due to rib-shortening and column concentrations. If the values of w_c and w_s include some uniformly distributed live load, the dead load stresses will be equal to the stresses produced by the total loading assumed minus the stresses produced by the uniform live load. The dead load moments will then be equal and opposite in sign to the moment produced by the live load assumed. It is possible to lay out the axis for the dead load only or for the dead load plus part of the live load, as may be desired. In the following formulas it will be assumed that the dead load only is used, although they may be used for either case.

The vertical reaction, V_d, is equal to one-half the total load on the rib, or,

$$V_d = \int_0^{l_1} w_x \, d\,x = l_1 \int_0^1 w_x \, d\,z$$

It has been proven that:

$$w_x = w_c \cosh z\,k,$$

hence,

$$V_d = w_c \, l_1 \int_0^1 \cosh z\,k \; d\,z$$

or,

$$V_d = w_c \, l_1 \, \frac{\sqrt{g^2 - 1}}{k} \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

As it was originally assumed that:

$$k^2 = l_1^2 \, \frac{w_c \, (g - 1)}{r \, H_d},$$

$$H_d = w_c \, \frac{l_1^2 \, (g - 1)}{r \, k^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The normal thrust at any point in the rib is:

$$T_d = \frac{H_d}{\cos \phi} = H_d \sqrt{1 + \tan^2 \phi} \quad \ldots\ldots\ldots\ldots\ldots (47)$$

TABLE 6.—Arch Axis Co-Ordinates.

$g = \frac{w_s}{w_c}$	N.	Values of $\frac{y_0}{r}$ (from Equation (38))										
		Point 0 (Springing line).	Point 1.	Point 2.	Point 3.	Point 4.	Point 5 (Quarter-point).	Point 6.	Point 7.	Point 8.	Point 9.	Point 10 (Crown).
1.000	0.250	1.000	0.8100	0.6400	0.4900	0.3600	0.2500	0.1600	0.0900	0.0400	0.0100	0.0000
1.167	0.245	1.000	0.8060	0.6339	0.4834	0.3539	0.2450	0.1563	0.0878	0.0390	0.0098	0.0000
1.347	0.240	1.000	0.8019	0.6277	0.4769	0.3478	0.2400	0.1527	0.0856	0.0380	0.0095	0.0000
1.543	0.235	1.000	0.7977	0.6214	0.4701	0.3416	0.2350	0.1493	0.0835	0.0370	0.0092	0.0000
1.756	0.230	1.000	0.7934	0.6151	0.4632	0.3353	0.2300	0.1458	0.0814	0.0360	0.0090	0.0000
1.987	0.225	1.000	0.7890	0.6087	0.4563	0.3293	0.2250	0.1424	0.0792	0.0350	0.0087	0.0000
2.240	0.220	1.000	0.7847	0.6022	0.4494	0.3229	0.2200	0.1386	0.0771	0.0340	0.0085	0.0000
2.514	0.215	1.000	0.7801	0.5957	0.4425	0.3167	0.2150	0.1351	0.0749	0.0330	0.0082	0.0000
2.814	0.210	1.000	0.7755	0.5891	0.4355	0.3104	0.2100	0.1315	0.0728	0.0320	0.0080	0.0000
3.141	0.205	1.000	0.7709	0.5824	0.4285	0.3041	0.2050	0.1281	0.0707	0.0310	0.0077	0.0000
3.500	0.200	1.000	0.7662	0.5757	0.4215	0.2978	0.2000	0.1245	0.0686	0.0300	0.0074	0.0000
3.893	0.195	1.000	0.7615	0.5689	0.4145	0.2914	0.1950	0.1209	0.0665	0.0290	0.0072	0.0000
4.324	0.190	1.000	0.7567	0.5621	0.4073	0.2851	0.1900	0.1176	0.0644	0.0281	0.0070	0.0000
4.801	0.185	1.000	0.7518	0.5551	0.4000	0.2787	0.1850	0.1140	0.0623	0.0271	0.0067	0.0000
5.321	0.180	1.000	0.7469	0.5481	0.3927	0.2723	0.1800	0.1106	0.0602	0.0262	0.0065	0.0000
5.898	0.175	1.000	0.7420	0.5410	0.3854	0.2659	0.1750	0.1072	0 0582	0.0252	0.0062	0.0000
6.536	0.170	1.000	0.7367	0.5337	0.3781	0.2595	0.1700	0.1037	0.0562	0.0243	0.0059	0.0000
7.244	0.165	1.000	0.7313	0.5264	0.3707	0.2531	0.1650	0.1002	0.0541	0.0233	0.0057	0.0000
8.031	0.160	1.000	0.7259	0.5190	0.3632	0.2466	0.1600	0.0968	0.0521	0.0224	0.0055	0.0000
8.906	0.155	1.000	0.7205	0.5116	0.3557	0.2399	0.1550	0.0934	0.0501	0.0215	0.0053	0.0000
9.889	0.150	1.000	0.7151	0.5040	0.3480	0.2332	0.1500	0.0901	0.0483	0.0206	0.0050	0.0000

TABLE 7.—INTERCEPTS TO DETERMINE POSITION OF TANGENTS TO ARCH AXIS.

$g = \frac{w_s}{w_c}$	N.	Values of $\frac{s}{l_1} = \frac{\cosh z\,k - 1}{k \sinh z\,k}$.										
		Point 0 (Springing line).	Point 1.	Point 2.	Point 3.	Point 4.	Point 5 (Quarter-point).	Point 6.	Point 7.	Point 8.	Point 9.	Point 10 (Crown).
1.000	0.25	0.5000	0.4500	0.4000	0.3500	0.3000	0.2500	0.2000	0.1500	0.1000	0.0500	0.0000
1.347	0.24	0.4743	0.4311	0.3865	0.3410	0.2943	0.2466	0.1982	0.1492	0.0998	0.0500	0.0000
1.756	0.23	0.4503	0.4129	0.3734	0.3319	0.2884	0.2432	0.1965	0.1485	0.0996	0.0500	0.0000
2.240	0.22	0.4279	0.3957	0.3607	0.3229	0.2825	0.2397	0.1946	0.1478	0.0993	0.0499	0.0000
2.814	0.21	0.4070	0.3792	0.3482	0.3140	0.2765	0.2360	0.1926	0.1469	0.0991	0.0499	0.0000
3.500	0.20	0.3872	0.3634	0.3360	0.3051	0.2704	0.2323	0.1905	0.1459	0.0988	0.0498	0.0000
4.324	0.19	0.3686	0.3482	0.3241	0.2962	0.2643	0.2285	0.1884	0.1449	0.0985	0.0498	0.0000
5.321	0.18	0.3510	0.3335	0.3125	0.2875	0.2583	0.2246	0.1862	0.1439	0.0983	0.0497	0.0000
6.536	0.17	0.3342	0.3194	0.3011	0.2787	0.2520	0.2206	0.1840	0.1429	0.0980	0.0497	0.0000
8.031	0.16	0.3182	0.3058	0.2899	0.2700	0.2457	0.2164	0.1817	0.1419	0.0976	0.0496	0.0000
9.889	0.15	0.3030	0.2926	0.2788	0.2613	0.2393	0.2121	0.1792	0.1407	0.0972	0.0496	0.0000

Table 9 may be used for the solution of Equations (45) and (46).

Numerical Example of Arch Axis Calculation.—The axis of the arch rib used in the numerical example of Section III will be computed and checked. It is assumed that the fill at the crown is 18 in. deep and that the roadway follows a parabolic curve, 6 in. higher at the crown than at the springing line. Over the fill is a pavement weighing 100 lb. per sq. ft.

The dimensions of the rib are, as before:

$$l = 100 \text{ ft.} \quad d_c = 18 \text{ in.}$$
$$r = 15 \text{ ft.} \quad d_s = 27 \text{ in.}$$
$$g = \frac{w_s}{w_c} = \frac{2\,371}{505} = 4.70$$

From Table 6, $N = 0.18604$. The ordinate of the arch axis is (Equation (38)):

$$y_0 = \frac{r}{g - 1} (\cosh z\,k - 1)$$
$$= \frac{15}{4.70 - 1} (\cosh z\,k - 1)$$

From a table of hyperbolic functions:

$$k = \cosh^{-1} g = 2.2292$$

Then the computation of y_0 is as shown in Table 8.

TABLE 8.—Values of y_0.

z.	$z\,k$.	cosh $z\,k$.	y_0, in feet.
0.1	0.2229	1.0250	0.101
0.2	0.4458	1.1011	0.410
0.3	0.6687	1.2321	0.942
0.4	0.8916	1.4245	1.72
0.5	1.1146	1.6883	2.79
0.6	1.3374	2.0358	4.20
0.7	1.5603	2.4852	6.03
0.8	1.7832	3.0585	8.35
0.9	2.0061	3.7845	11.30
1.0	2.2292	4.7000	15.00

For the vertical reaction due to the dead load, Table 9 gives for a rib, 1 ft. wide:

$$V_d = 1.0319\ l\,w_c$$
$$= 1.0319 \times 100 \times 505 = 52\,111 \text{ lb.}$$

and for the horizontal thrust:

$$H_d = 0.1863 \frac{w_c\ l^2}{r}$$
$$= 0.1863 \frac{505 \times 10\,000}{15} = 62\,720 \text{ lb.}$$

As a check on these results, the same quantities are computed, using the actual loads on the arch. The thicknesses of rib used are those given in the example of Section III; they were computed by the method to be presented in

Section V. The weight of the concrete is taken at 150 lb. per cu. ft. and that of the fill at 120 lb. per cu. ft. The span was divided into twenty sections and the dead load of each section computed. These loads were applied at the center of gravity of each section, and the pressure line was computed for a rise of 15 ft. The weights are given in Table 10. The weight of the pavement is 500 lb. per section.

TABLE 9.—VALUES OF N, g, H_d, V_d, tan ϕ_s.

N.	$g = \frac{w_s}{w_c}$ (Equation (41)).	$C_d = \frac{r H_d}{w_c l^2}$ (Equation (46)).	$\frac{V_d}{w_c l}$ (Equation (45)).	$\frac{l}{r}$ tan ϕ_s (Equation (43)).
0.250	1.000	0.1250	0.5000	4.000
0.245	1.166	0.1285	0.5277	4.120
0.240	1.347	0.1320	0.5566	4.217
0.235	1.543	0.1358	0.5875	4.328
0.230	1.756	0.1397	0.6206	4.442
0.225	1.988	0.1438	0.6554	4.556
0.220	2.240	0.1483	0.6933	4.675
0.215	2.515	0.1530	0.7331	4.798
0.210	2.814	0.1579	0.7761	4.915
0.205	3.141	0.1632	0.8219	5.030
0.200	3.500	0.1687	0.8713	5.165
0.195	3.893	0.1746	0.9242	5.294
0.190	4.324	0.1808	0.9812	5.427
0.185	4.799	0.1875	1.0430	5.561
0.180	5.321	0.1946	1.1092	5.700
0.175	5.897	0.2022	1.1874	5.841
0.170	6.536	0.2104	1.2593	5.985
0.165	7.244	0.2193	1.3443	6.132
0.160	8.031	0.2287	1.4370	6.288
0.155	8.910	0.2389	1.5382	6.440
0.150	9.889	0.2499	1.6496	6.601

It may be noted that the computed value of V_d checks the value given by Equation (45) almost exactly. The horizontal thrust and the pressure-line ordinates are found by taking moments about the center point of the rib at mid-span. The values of these moments and the computed ordinates are given in Table 11. Column (7) gives the ordinates of the axis as computed from Equation (38) for comparison.

TABLE 10.—DEAD LOADS ON ARCH.

Section.	Height of fill, in feet.	Weight of fill, in pounds.	Vertical thickness of rib, in feet.	Weight of rib, in pounds.	Total weight per section, in pounds.
1	13.60	8 170	2.711	2 032	10 702
2	10.54	6 330	2.355	1 765	8 595
3	8.004	4 830	2.116	1 585	6 915
4	6.185	3 710	1.944	1 458	5 668
5	4.688	2 810	1.823	1 367	4 677
6	3.541	2 125	1.733	1 300	3 925
7	2.698	1 620	1.656	1 241	3 861
8	2.121	1 273	1.598	1 198	2 971
9	1.743	1 046	1.551	1 162	2 708
10	1.548	930	1.515	1 136	2 566
$\Sigma W = V_d$					= 52 088
By Equation (45), V_d					= 52 111

The horizontal thrust is equal to the moment of the vertical reaction minus the moment of the dead loads divided by the rise, or,

$$H_d = \frac{(50 \times 52\,088) - 1\,660\,720}{15} = 62\,850 \text{ lb.}$$

By Equation (46),

$$H_d = 62\,720 \text{ lb.}$$

TABLE 11.—MOMENTS, WEIGHTS, AND ORDINATES.

Section. (1)	$\bar{x}$ (2)	M (3)	ΣM (4)	ΣW (5)	y_0 (6)	y_0 (Equation (38)) (7)
1	47.60	509 500	1 660 720	52 088	15.00	15.00
2	42.59	366 000	1 151 220	41 386	11.31	11.30
3	37.59	260 000	785 220	32 791	8.38	8.35
4	32.58	184 600	525 220	25 876	6.06	6.03
5	27.58	129 000	340 620	20 208	4.23	4.20
6	22.56	88 500	211 620	15 531	2.81	2.79
7	17.55	59 000	123 120	11 606	1.74	1.72
8	12.54	37 250	64 120	8 245	0.95	0.94
9	7.53	20 400	26 870	5 274	0.41	0.41
10	2.52	6 470	6 470	2 566	0.10	0.10

The pressure-line ordinate at division points was computed by subtracting the moment of the loads between that point and the center of the span from the moment of the vertical shear at the section and dividing by the horizontal thrust. For instance for Point 1:

$$y_0 = \frac{(45 \times 41\,386) - 1\,151\,220}{62\,850} = 11.31 \text{ ft.}$$

These computations, made with a slide-rule, show a maximum variation of only $\frac{3}{8}$ in. between the arch axis computed by the two different methods for a 100-ft. span. Considering the many uncertainties involved in arch design and construction, a closer agreement could not be desired.

The method for determining the arch axis given by J. A. L. Waddell, M. Am. Soc. C. E.,* and by Melan,† which assumes a parabolic variation of loading on the arch, gives an ordinate at the quarter-point for this arch of 2.68 ft., or $1\frac{9}{16}$ in. above the pressure line as calculated, introducing some dead load moment. The latter method gives quite close results for lower values of g, that is, for open spandrel arches with a more nearly uniform loading; the formulas as developed herein will also be accurate for all values of g, and may be used for filled arches as well as for open spandrel arches.

V.—FORM OF ARCH RIB

In Section IV is given a method of establishing the center line of the arch rib. As the next step it is necessary to determine the thickness of the rib and its moment of inertia at all points. A formula for the variation of the rib

* "Bridge Engineering", Vol. I.

† "Handbuch für Eisenbetonbau", Vol. I.

thickness must be selected which will give a shape satisfactory for practical design and at the same time must be suitable for simple mathematical analysis. As the term, dw, or $\frac{ds}{EI}$, is used in the solution of the arch, it is desirable to use a formula for the thickness of rib which will simplify the integrations.

The expression for dw is:

$$dw = \frac{ds}{EI} = \frac{dx}{EI\cos\phi} = \frac{l_1\,dz}{EI\cos\phi}$$

This is difficult to integrate because of the involved expression for $\cos\phi$, and the solution will be greatly simplified by making the value of I inversely proportional to $\cos\phi$ so that that term can be eliminated. Then,

$$\frac{I_x}{I_c} = \frac{1}{f(z)\cos\phi}$$

in which,

I_c = moment of inertia at the crown;
I_x = moment of inertia at any section; and
$f(z)$ = a function of z to be determined.

At the crown,

$$\frac{I_c}{I_c} = \frac{1}{f(z)\cos\phi} = 1, \text{ or } f(z) = 1$$

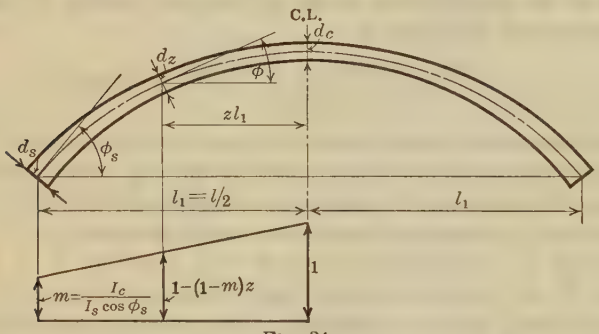


FIG. 24

At the springing line,

$$\frac{I_s}{I_c} = \frac{1}{f(z)\cos\phi_s}, \text{ or } f(z) = \frac{I_c}{I_s\cos\phi_s}$$

Then, assuming a straight-line variation of $f(z)$ between the crown and the springing line as shown in Fig. 24:

$$f(z) = 1 - \left(1 - \frac{I_c}{I_s\cos\phi_s}\right)z = 1 - (1-m)z$$

in which,

$$m = \frac{I_c}{I_s\cos\phi_s}$$

Therefore,

$$\frac{I_x}{I_c} = \frac{1}{[1 - (1 - m)\, z] \cos \phi}$$

or,

$$\frac{I_c}{I_x \cos \phi} = 1 - (1 - m)\, z \qquad (48)$$

It will be assumed that the moment of inertia of the rib varies as the cube of the rib thickness, hence,

$$\frac{I_c}{I_x} = \left(\frac{d_c}{d_x}\right)^3$$

and,

$$\left(\frac{d_c}{d_x}\right)^3 = [1 - (1 - m)\, z] \cos \phi$$

then,

$$d_x = d_c \frac{1}{\sqrt[3]{[1 - (1 - m)\, z] \cos \phi}}$$

By substituting $\cos \phi = \frac{1}{\sqrt{1 + \tan^2 \phi}}$, the formula reduces to:

$$d_x = d_c\, c \sqrt[3]{1 + \tan^2 \phi} \qquad (49)$$

in which,

$$c = \frac{1}{\sqrt[3]{1 - (1 - m)\, z}} \qquad (49a)$$

It was assumed that:

$$m = \frac{I_c}{I_s \cos \phi_s} \qquad (50)$$

For plain rectangular cross-sections:

$$m = \frac{d_c^3}{d_s^3 \cos \phi_s} \qquad (51)$$

When the reinforcement of the rib is such that the values of m given by Equations (50) and (51) do not agree, the one derived from Equation (50) should be used in the formulas to be developed for the thrusts and moments, but that given by Equation (51) must be used for computing the thickness of rib by Equation (49). Thus, although the variation expressed by Equation (48) will not be exactly true, it will have no appreciable effect on the accuracy of the values of the thrusts and moments.

The term, m, will be used to designate the form of the arch rib and the term, N, to designate the form of the axis of the rib. These two values together with the span, rise, and thickness or moment of inertia of the rib at the crown, entirely establish the rib and its reactions under live load and change of temperature.

The variation of rib thickness given by Equation (49) has been found to be satisfactory in practical bridge design. In order to simplify the solution of this formula, Tables 12 and 13 give the values of $\tan^2 \phi$ and c. The proper values can be obtained by interpolation.

TABLE 12.—DETERMINATION OF RIB THICKNESS—VALUES OF $\frac{l^2}{r^2}\ \tan^2\phi$.

$g = \frac{w_s}{w_c}$.	N.	VALUES OF $\frac{l^2}{r^2}\ \tan^2\phi$ (FROM EQUATION (42)).										
		Point 0 (Springing line).	Point 1.	Point 2.	Point 3.	Point 4.	Point 5 (Quarter-point).	Point 6.	Point 7.	Point 8.	Point 9.	Point 10 (Crown).
1.000	0.25	16.000	12.960	10.240	7.840	5.760	4.000	2.560	1.440	0.640	0.160	0
1.347	0.24	17.780	13.846	10.552	7.821	5.583	3.788	2.377	1.316	0.580	0.145	0
1.756	0.23	19.722	14.769	10.852	7.791	5.403	3.578	2.200	1.199	0.523	0.130	0
2.240	0.22	21.841	15.730	11.149	7.748	5.220	3.371	2.029	1.088	0.468	0.116	0
2.814	0.21	24.150	16.731	11.443	7.694	5.034	3.166	1.864	0.983	0.417	0.102	0
3.500	0.20	26.676	17.780	11.734	7.629	4.844	2.964	1.706	0.883	0.369	0.089	0
4.324	0.19	29.441	18.885	12.020	7.552	4.650	2.765	1.554	0.789	0.325	0.076	0
5.321	0.18	32.476	20.047	12.297	7.463	4.450	2.569	1.408	0.701	0.284	0.065	0
6.536	0.17	35.812	21.269	12.564	7.360	4.243	2.376	1.268	0.619	0.246	0.056	0
8.031	0.16	39.494	22.553	12.820	7.238	4.029	2.187	1.135	0.541	0.211	0.048	0
9.889	0.15	43.572	23.899	13.063	7.095	3.809	2.001	1.008	0.467	0.180	0.041	0

TABLE 13.—Determination of Rib Thickness—Values of c.

Value of $c = \dfrac{1}{\sqrt[3]{1-(1-m)z}}$ (From Equation (49 a)).

m.	Point 0 (Springing line). $z = 1.0$	Point 1. $z = 0.9$	Point 2. $z = 0.8$	Point 3. $z = 0.7$	Point 4. $z = 0.6$	Point 5 (Quarter-point). $z = 0.5$	Point 6. $z = 0.4$	Point 7. $z = 0.3$	Point 8. $z = 0.2$	Point 9. $z = 0.1$	Point 10 (Crown). $z = 0.0$
1.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.8	1.077	1.068	1.060	1.052	1.044	1.036	1.028	1.021	1.014	1.007	1.000
0.6	1.186	1.160	1.137	1.116	1.096	1.077	1.060	1.044	1.028	0.014	1.000
0.5	1.260	1.221	1.186	1.154	1.126	1.101	1.077	1.056	1.036	1.017	1.000
0.4	1.357	1.295	1.244	1.199	1.160	1.126	1.096	1.068	1.044	1.021	1.000
0.3	1.494	1.393	1.315	1.252	1.199	1.154	1.116	1.082	1.052	1.024	1.000
0.25	1.587	1.454	1.357	1.282	1.221	1.170	1.126	1.089	1.056	1.026	1.000
0.20	1.710	1.529	1.405	1.315	1.244	1.186	1.137	1.096	1.060	1.028	1.000
0.15	1.882	1.621	1.462	1.352	1.268	1.203	1.149	1.103	1.064	1.030	1.000

In determining the rib thickness, it is possible to compute the value of m after both d_c and d_s are assumed, or it is possible first to assume m and d_c and compute d_s from them by the formula:

$$d_s = \frac{d_c}{\sqrt[3]{m \cos \phi_s}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

The value of $\cos \phi_s$ may be computed from the formula:

$$\cos \phi_s = \frac{1}{\sqrt{1 + \tan^2 \phi_s}}$$

with the value of $\tan^2 \phi_s$ obtained from Equation (43), from Table 12, or from the following approximate formula (Equation (53)):

$$\tan^2 \phi_s = \left(\frac{r}{l}\right)^2 (11.50 + 4.55\, g - 0.13\, g^2) \quad \ldots\ldots\ldots\ldots (53)$$

Numerical Example.—Thickness of Rib.—The following calculations give the thickness of the rib which has been used in the previous numerical examples. (See Fig. 19.)

As before:

$l = 100$ ft. $\qquad d_c = 18$ in.
$r = 15$ ft. $\qquad d_s = 27$ in.
$g = 4.70 \qquad \tan \phi_s = 0.8304$
$N = 0.18604 \qquad \cos \phi_s = 0.769$

then,

$$m = \frac{18^3}{27^3 \times 0.769} = 0.385$$

$$\tan \phi = \frac{2\, r\, k}{l\,(g - 1)} \sinh z\, k = 0.1808 \sinh z\, k$$

$$k = \cosh^{-1} g = 2.2292$$

Using these values, the various steps in the solution of Equations (49) and (49*a*) are given in Table 14.

TABLE 14.—Computations for Thickness of Arch Rib.

Point.	z	$z\,k$	$\sinh z\,k$	$\tan \phi$	$1 + \tan^2 \phi$	$\sqrt[6]{1 + \tan^2 \phi}$	$\frac{(1-m)}{z}$	$\frac{1}{\sqrt[3]{1-(1-m)z}}$	d_z
0	1.0	2.2292	4.5925	0.8300	1.688	1.092	0.6150	1.372	27.00
1	0.9	2.0061	3.6499	0.6600	1.435	1.063	0.5535	1.307	25.00
2	0.8	1.7832	2.8904	0.5225	1.273	1.042	0.4920	1.252	23.50
3	0.7	1.5603	2.2751	0.4115	1.169	1.028	0.4305	1.207	22.35
4	0.6	1.3374	1.7733	0.3207	1.103	1.017	0.3690	1.165	21.33
5	0.5	1.1146	1.3602	0.2460	1.0655	1.012	0.3075	1.130	20.60
6	0.4	0.8916	1.0145	0.1835	1.0337	1.006	0.2460	1.098	19.90
7	0.3	0.6687	0.7197	0.1301	1.0169	1.003	0.1845	1.069	19.30
8	0.2	0.4458	0.4607	0.0833	1.0069	1.001	0.1230	1.043	18.80
9	0.1	0.2229	0.2248	0.0407	1.0016	1.000	0.0615	1.020	18.35
10	0.0	.....	.....	.....	.....	.....	.....	.....	18.00

This calculation is much simpler when the values of $\tan^2 \phi$ and c are taken from Tables 12 and 13.

VI.—Solution of Arch Formulas

The formulas for thrusts and moments developed in Section II can be solved by substituting in them the formulas for the shape of the axis and rib as given in Sections IV and V.

Value of $d\,w$.—

$$d\,w = \frac{d\,s}{E\,I} \text{ or } \frac{d\,s}{E\,I_x}$$

$$d\,s = \frac{d\,x}{\cos\,\phi} \text{ and } d\,x = l_1\,d\,z$$

hence,

$$d\,w = \frac{l_1\,d\,z}{E\,I_x \cos\,\phi}$$

Equation (48) gives:

$$\frac{I_c}{I_x \cos\,\phi} = 1 - (1 - m)\,z$$

or,

$$\frac{1}{I_x} = \frac{[1 - (1 - m)\,z] \cos\,\phi}{I_c}$$

Substituting this in the above equation for $d\,w$:

$$d\,w = \frac{l_1\,[1 - (1 - m)\,z] \cos\,\phi\,d\,z}{E\,I_c \cos\,\phi}$$

or,

$$d\,w = \frac{l_1}{E\,I_c}\,[1 - (1 - m)\,z]\,d\,z \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

Equation for Elastic Line.—The differential equation for the elastic line is (Equation (5)):

$$\frac{d^2\,\delta}{d\,x^2} = -\frac{M}{E\,I_x \cos\,\phi}$$

or,

$$\frac{d^2\,\delta}{d\,z^2} = -\frac{M\,l_1^2}{E\,I_x \cos\,\phi}$$

Combining this with Equation (48),

$$\frac{d^2\,\delta}{d\,z^2} = -\frac{M\,l_1^2}{E\,I_c}\,[1 - (1 - m)\,z] \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

Determination of M'.—The value of M' as given by Equation (28) is:

$$M' = P\,\frac{\delta_{MA}}{\sum_L^R \Delta\,w} = P\,\frac{\delta_{MA}}{\int_L^R d\,w}$$

For a unit load, $P = 1$:

$$M' = \frac{\delta_{MA}}{\int_L^R d\,w}$$

The denominator can be obtained by direct integration of Equation (54):

$$\int_L^R d\,w = 2\int_0^1 \frac{l_1}{E\,I_c}\,[1-(1-m)\,z]\,d\,z$$

$$= \frac{l}{E\,I_c}\left[\int_0^1 d\,z - (1-m)\int_0^1 z\,d\,z\right]$$

$$= \frac{l}{E\,I_c}\left[1-(1-m)\,\frac{1}{2}\right]$$

$$\int_L^R d\,w = \frac{l}{E\,I_c}\,\frac{1+m}{2} \qquad (56)$$

The numerator, δ_{MA}, is the ordinate of the elastic line for the rib when $M_L = 1$ (see Fig. 12). In that case, the moment at any point in the rib is $M = 1$, and the ordinate of the elastic line can be obtained by integrating twice the differential equation for the elastic line.

Equation (55) is:

$$\frac{d^2\,\delta}{d\,z^2} = -\frac{M\,l_1^2}{E\,I_c}\,[1-(1-m)\,z]$$

Then, placing $M = 1$ and changing the sign, because the δ required is to be measured from the tangent and not from the chord of the elastic line,

$$\frac{d^2\,\delta_{MA}}{d\,z^2} = \frac{l^2}{4\,E\,I_c}\,[1-(1-m)\,z]$$

Integrating,

$$\frac{d\,\delta_{MA}}{d\,z} = \frac{l^2}{4\,E\,I_c}\left[z-(1-m)\,\frac{z^2}{2}+C_1\right]$$

When

$$z = 1,\ \frac{d\,\delta_{MA}}{d\,z} = 0;\ \text{ therefore, } C_1 = -\frac{1+m}{2}$$

then,

$$\frac{d\,\delta_{MA}}{d\,z} = \frac{l^2}{4\,E\,I_c}\left[z-(1-m)\,\frac{z^2}{2}-\frac{1+m}{2}\right]$$

Integrating again,

$$\delta_{MA} = \frac{l^2}{4\,E\,I_c}\left[\frac{z^2}{2}-(1-m)\,\frac{z^3}{6}-\left(\frac{1+m}{2}\right)z+C_2\right].$$

When

$$z = 1,\ \delta_{MA} = 0;\ \text{ therefore, } C_2 = \frac{1}{6}+\frac{m}{3}$$

then,

$$\delta_{MA} = \frac{l^2}{4\,E\,I_c}\left[\frac{z^2}{2}-(1-m)\,\frac{z^3}{6}-\left(\frac{1+m}{2}\right)z+\frac{1}{6}+\frac{m}{3}\right]$$

$$= \frac{l^2}{24\,E\,I_c}\,[-(1-m)\,z^3+3\,z^2-3\,z\,(1+m)+(1+2\,m)]$$

This equation applies only to the right-hand half of the rib and must be modified for application to the left half. For the left side, this equation would

give only that part of the deflection marked b in Fig. 25. As the angle, θ_{ML}, is always extremely small,

$$a = \theta_{ML}\, z\, l_1$$

And as,

$$\theta_{ML} = \int_L^R M\, d\, w = \int_L^R d\, w = \frac{l}{E\, I_c}\,\frac{1+m}{2}$$

$$a = \frac{z\, l^2}{2\, E\, I_c}\cdot\frac{1+m}{2} = z\,\frac{l^2}{24\, E\, I_c}\,(6+6\,m)$$

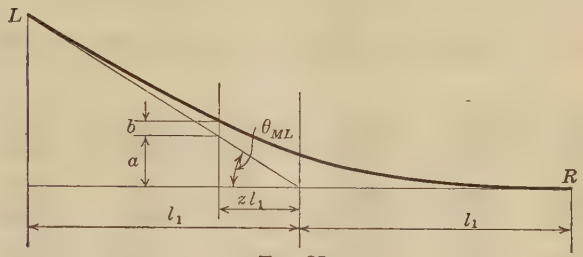


FIG. 25.

Then adding this value of a to the value of δ_{MA} found for the right-hand side, there results for the left-hand side:

$$\delta_{MA} = \frac{l^2}{24\, E\, I_c}\,[-(1-m)\, z^3 + 3\, z^2 + 3\, z\,(1+m) + (1+2\,m)]$$

Then to the left of the center line:

$$M' = \frac{\delta_{MA}}{\int_L^R d\, w} = \frac{l}{12\,(1+m)}\,[(m-1)\, z^3 + 3\, z^2 + 3\, z\,(1+m) + (1+2\,m)] \quad \ldots\ldots (57)\ \text{(left)}$$

To the right of the center line:

$$M' = \frac{l}{12\,(1+m)}\,[(m-1)\, z^3 + 3\, z^2 - 3\, z\,(1+m) + (1+2\,m)] \quad \ldots\ldots (57)\ \text{(right)}$$

Table 15 gives the solution of these equations.

Numerical Example.—Applying Equations (56) and (57) to the rib used in the previous examples:

$l = 100$ ft. $= 1\,200$ in. $\qquad I_c = 6\,399$ in.^{4}

$r = 15$ ft. $= 180$ in. $\qquad I_s = 24\,560$ in.^{4}

$$\cos\phi_s = 0.769$$

$$m = \frac{I_c}{I_s \cos\phi_s} = 0.339$$

$$\int_L^R d\, w = \frac{l}{E\, I_c}\cdot\frac{1+m}{2} = \frac{1\,200 \times 1.339}{2 \times 2\,000\,000 \times 6\,399} = 0.0000000628$$

The value of $\sum_L^R \Delta\, w$ computed in the example in Section III is:

$$\sum_L^R \Delta\, w = 2.601 \times 2 \times \frac{12}{10^9} = 0.0000000624$$

For a unit load at the center of the span, $z = 0$, and Equation (57) reduces to:

$$M' = \frac{l}{12\,(1+m)}\,(1+2\,m) = \frac{1\,200 \times 1.678}{12 \times 1.339} = 125.3 \text{ in-lb.} = 10.44 \text{ ft-lb.}$$

The graphical solution of Section III gave:

$$M' = 10.43 \text{ ft-lb.}$$

TABLE 15.—VALUES OF $\frac{M'}{l}$ (BY EQUATION (57) FOR ALL VALUES OF g).

Point loaded.	$m = 0.15$.	$m = 0.20$.	$m = 0.25$.	$m = 0.30$.	$m = 0.35$.	$m = 0.40$.	$m = 0.50$.
0	0.50000	0.50000	0.50000	0.50000	0.50000	0.50000	0.50000
1	0.45039	0.45047	0.45055	0.45062	0.45069	0.45075	0.45086
2	0.40180	0.40211	0.40240	0.40267	0.40291	0.40314	0.40356
3	0.35460	0.35525	0.35585	0.35641	0.35692	0.35739	0.35825
4	0.30916	0.31022	0.31120	0.31210	0.31294	0.31371	0.31511
5	0.26585	0.26736	0.26875	0.27003	0.27122	0.27232	0.27431
6	0.22505	0.22700	0.22880	0.23046	0.23200	0.23339	0.23600
7	0.18711	0.18947	0.19165	0.19366	0.19552	0.19725	0.20036
8	0.15241	0.15511	0.15760	0.15990	0.16203	0.16400	0.16756
9	0.12131	0.12425	0.12695	0.12944	0.13176	0.13389	0.13775
10	0.09420	0.09722	0.10000	0.10257	0.10494	0.10714	0.11111
9′	0.07131	0.07425	0.07695	0.07944	0.08176	0.08389	0.08775
8′	0.05241	0.05511	0.05760	0.05990	0.06203	0.06400	0.06756
7′	0.03711	0.03947	0.04165	0.04366	0.04552	0.04725	0.05036
6′	0.02505	0.02700	0.02880	0.03046	0.03200	0.03339	0.03600
5′	0.01585	0.01736	0.01875	0.02003	0.02122	0.02232	0.02131
4′	0.00916	0.01022	0.01120	0.01210	0.01294	0.01371	0.01511
3′	0.00460	0.00525	0.00585	0.00641	0.00692	0.00739	0.00825
2′	0.00180	0.00211	0.00240	0.00267	0.00291	0.00314	0.00356
1′	0.00039	0.00047	0.00055	0.00062	0.00069	0.00075	0.00086
0′	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Determination of V_L.—From Equation (27), when $P = 1$:

$$V_L = P\,\frac{\delta_{VA}}{\delta_{VL}} = \frac{\delta_{VA}}{\delta_{VL}}$$

Both the numerator and the denominator of the expression for V_L are obtained for the elastic line of the arch when $V_L = 1$, as shown in Fig. 11. In this case, the moment at any point is:

$$M = V_L\,x = x = \frac{z\,l}{2}$$

Then the differential equation for the elastic line is:

$$\frac{d^2\,\delta}{d\,z^2} = \frac{M\,l}{4\,E\,I_c}\,[1 - (1 - m)\,z]$$

$$\frac{d^2\,\delta_{VA}}{d\,z^2} = \frac{l^2}{8\,E\,I_c}\,[z - (1 - m)\,z^2]$$

After integrating once:

$$\frac{d\,\delta_{VA}}{d\,z} = \frac{l^2}{8\,E\,I_c}\left[\frac{z^2}{2} - (1 - m)\,\frac{z^3}{3} + C_1\right]$$

When

$$z = 1,\ \frac{d\,\delta_{VA}}{d\,z} = 0;\ \text{therefore, } C_1 = -\frac{1 + 2\,m}{6}$$

then,

$$\frac{d\,\delta_{VA}}{d\,z} = \frac{l^2}{8\,E\,I_c}\left[\frac{z^2}{2} - (1 - m)\,\frac{z^3}{3} - \frac{1 + 2\,m}{6}\right]$$

Integrating again,

$$\delta_{VA} = \frac{l^2}{8\,E\,I_c}\left[\frac{z^3}{6} - (1 - m)\,\frac{z^4}{12} - \left(\frac{1 + 2\,m}{6}\right) z + C_2\right]$$

When

$$z = 1,\ \delta_{VA} = 0;\ \text{therefore, } C_2 = \frac{1 + 3\,m}{12}$$

then,

$$\delta_{VA} = \frac{l^2}{8\,E\,I_c}\left[\frac{z^3}{6} - (1 - m)\,\frac{z^4}{12} - \left(\frac{1 + 2\,m}{6}\right) z + \frac{1 + 3\,m}{12}\right]$$

$$= \frac{l^2}{96\,E\,I_c}\left[(m - 1)\,z^4 + 2\,z^3 - (2 + 4\,m)\,z + (1 + 3\,m)\right]$$

When

$$z = 0,\ \delta_{VA} = \frac{l^2}{96\,E\,I_c}\,(1 + 3\,m)$$

From symmetry, δ_{VL} is twice this value of δ_{VA} at the center of the span, so that:

$$\delta_{VL} = \frac{l^2}{48\,E\,I_c}\,(1 + 3\,m)$$

This equation for δ_{VA} was derived for positive values of z and applies only to the right-hand side of the rib. To the right of the center line, the value of V_L will be:

$$V_L = \frac{\delta_{VA}}{\delta_{VL}} = \frac{1}{2\,(1 + 3\,m)}\left[(m - 1)\,z^4 + 2\,z^3 - (2 + 4\,m)\,z + (1 + 3\,m)\right] \quad \ldots\ldots (58)\ \text{(right)}$$

For the left side of the span, the value of V_L will be one minus the corresponding value on right side, or,

$$V_L = \frac{1}{2\,(1 + 3\,m)}\left[(1 - m)\,z^4 - 2\,z^3 + (2 + 4\,m)\,z + (1 + 3\,m)\right] \quad \ldots\ldots (58)\ \text{(left)}$$

Values of V_L from these equations are given in Table 16.

Numerical Example.—Using the same arch rib as before, the value of V_L for a load at the quarter-point is calculated as follows:

$$m = 0.339;\ z = 0.5$$

From Equation (58) (right):

$$V_L = \frac{1}{2 \times 2.017}\left[-\frac{0.661}{16} + \frac{2}{8} - \frac{3.356}{2} + 2.017\right] = 0.136$$

From Equation (58) (left):

$$V_L = \frac{1}{2 \times 2.017}\left[+\frac{0.661}{16} - \frac{2}{8} + \frac{3.356}{2} + 2.017\right] = 0.864$$

These values agree exactly with those obtained in Section III by the graphical method.

Determination of H and y_c.—The location of the horizontal axis through the elastic center of gravity of the rib from which the value of y is measured is given by the expressions:

$$b = \frac{\int y'\, d\, w}{\int d\, w}, \text{ or } y_c = \frac{\int y_0\, d\, w}{\int d\, w}$$

TABLE 16.—Vertical Reactions. Value of V_L for Unit Loads (From Equation (58), for All Values of g).

Point loaded.	$m = 0.15$.	$m = 0.20$.	$m = 0.25$.	$m = 0.30$.	$m = 0.35$.	$m = 0.40$.	$m = 0.50$.
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	0.9966	0.9960	0.9955	0.9951	0.9948	0.9945	0.9939
2	0.9844	0.9825	0.9810	0.9797	0.9786	0.9777	0.9762
3	0.9614	0.9582	0.9555	0.9532	0.9512	0.9496	0.9468
4	0.9268	0.9222	0.9185	0.9153	0.9126	0.9103	0.9065
5	0.8804	0.8750	0.8704	0.8668	0.8636	0.8608	0.8562
6	0.8220	0.8164	0.8119	0.8079	0.8047	0.8018	0.7971
7	0.7525	0.7476	0.7434	0.7399	0.7369	0.7344	0.7301
8	0.6741	0.6704	0.6672	0.6645	0.6622	0.6603	0.6565
9	0.5890	0.5869	0.5852	0.5837	0.5825	0.5814	0.5796
10	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000	0.5000
9′	0.4110	0.4131	0.4148	0.4163	0.4175	0.4186	0.4204
8′	0.3259	0.3296	0.3328	0.3355	0.3378	0.3397	0.3435
7′	0.2475	0.2524	0.2566	0.2601	0.2631	0.2656	0.2699
6′	0.1780	0.1836	0.1881	0.1921	0.1953	0.1982	0.2029
5′	0.1196	0.1250	0.1296	0.1332	0.1364	0.1392	0.1438
4′	0.0732	0.0778	0.0815	0.0847	0.0874	0.0897	0.0935
3′	0.0386	0.0418	0.0445	0.0468	0.0488	0.0504	0.0582
2′	0.0156	0.0175	0.0190	0.0203	0.0214	0.0223	0.0238
1′	0.0034	0.0040	0.0045	0.0049	0.0052	0.0055	0.0061
0′	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

The value of $\int d\, w$ has already been determined and is (Equation (56)):

$$\int_L^R d\, w = \frac{l}{E\, I_c} \cdot \frac{1 + m}{2}$$

The value of $\int y_0\, d\, w$ can be obtained by integrating the equation of the arch axis (Equation (38)), which is:

$$y_0 = \frac{r}{g - 1}(\cosh z\, k - 1)$$

Equation (54) is:

$$d\, w = \frac{l_1}{E\, I_c}[1 - (1 - m)\, z]\, d\, z$$

then,

$$\int_L^R y_0\, d\,w = \frac{2\, l_1\, r}{E\, I_c\, (g-1)} \int_0^1 (\cosh z\, k - 1)\, [1 - (1-m)\, z]\, d\, z$$

$$= \frac{l\, r}{E\, I_c\, (g-1)} \left[\int_0^1 (\cosh z\, k - 1)\, d\, z - (1-m) \int_0^1 (z \cosh z\, k - z)\, d\, z \right]$$

$$= \frac{l\, r}{E\, I_c\, (g-1)} \left[\frac{\sqrt{g^2-1}}{k} - 1 - (1-m) \left(\frac{\sqrt{g^2-1}}{k} - \frac{g-1}{k^2} - \frac{1}{2} \right) \right]$$

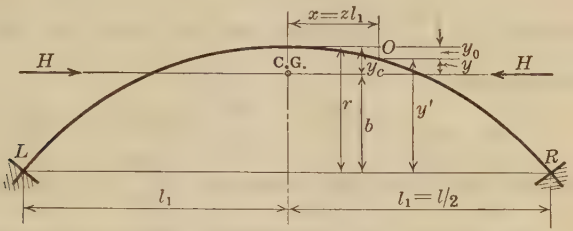


FIG. 26.

then the equation for y_c is:

$$y_c = \frac{\int y_0\, d\, w}{\int d\, w} = \frac{2\, E\, I_c}{l\, (1+m)} \int y_0\, d\, w$$

or,

$$y_c = \frac{2\, r}{(g-1)\, (1+m)} \left[\frac{\sqrt{g^2-1}}{k} - 1 \right.$$

$$\left. - (1-m) \left(\frac{\sqrt{g^2-1}}{k} - \frac{g-1}{k^2} - \frac{1}{2} \right) \right] \quad \ldots\ldots\ldots\ldots (59)$$

The values of y_c from Equation (59) are given in Table 17.

Both y_0 and y_c have now been determined, and y is given by the formula:

$$y = y_c - y_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (60)$$

TABLE 17.—VALUE OF y_c (BY EQUATION (59)).

N.	g.	VALUE OF $\frac{y_c}{r}$.					
		$m = 0.15$.	$m = 0.20$.	$m = 0.25$.	$m = 0.30$.	$m = 0.40$	$m = 0.50$.
0.25	1.000	0.2101	0.2222	0.2333	0.2436	0.2619	0.2778
0.24	1.347	0.2044	0.2163	0.2273	0.2374	0.2556	0.2713
0.23	1.756	0.1985	0.2103	0.2212	0.2312	0.2491	0.2647
0.22	2.240	0.1926	0.2043	0.2150	0.2250	0.2427	0.2580
0.21	2.814	0.1867	0.1983	0.2089	0.2187	0.2362	0.2513
0.20	3.500	0.1808	0.1922	0.2027	0.2124	0.2296	0.2446
0.19	4.324	0.1748	0.1861	0.1964	0.2060	0.2230	0.2378
0.18	5.321	0.1688	0.1799	0.1901	0.1996	0.2164	0.2309
0.17	6.536	0.1628	0.1738	0.1838	0.1931	0.2097	0.2240
0.16	8.031	0.1567	0.1675	0.1774	0.1865	0.2029	0.2170
0.15	9.889	0.1506	0.1612	0.1709	0.1799	0.1960	0.2099

For the value of the horizontal thrust, H, Equation (20a) gives:

$$H = \frac{P \int_A^R x' \, y \, d \, w}{\int_L^R y^2 \, d \, w \, . \, (1 + u)}$$

which for a unit load, $P = 1$, equals:

$$H = \frac{\delta_{HA}}{\delta_{HL} \, (1 + u)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (61)$$

The value of $\delta_{HL} = \int_L^R y^2 \, d \, w$ will first be found (see Fig. 10), using Equations (38) and (60):

$$y = y_c - y_0 = r \left[\frac{y_c}{r} - \frac{1}{g - 1} (\cosh z \, k - 1) \right] \ldots\ldots\ldots\ldots (62)$$

To simplify the solution of the expression, $\int_L^R y^2 \, d \, w$, use in its place, $\int_L^R y \, (y - a) \, d \, w$, in which a is any constant. This is true because,

$$\int_L^R y \, (y - a) \, d \, w = \int_L^R y^2 \, d \, w - a \int_L^R y \, d \, w$$

As,

$$a \int_L^R y \, d \, w = 0$$

then,

$$\int_L^R y^2 \, d \, w = \int_L^R y \, (y - a) \, d \, w$$

hence, a is equal to the sum of all the constant terms in the formula for y, which are not dependent on the value of z, and,

$$y - a = - \frac{r}{g - 1} \cosh z \, k$$

Then,

$$\int_L^R y^2 \, d \, w = - \frac{l \, r^2}{E \, I_c \, (g - 1)} \int_0^1 \left[\frac{y_c}{r} - \frac{1}{g - 1} (\cosh z \, k - 1) \right]$$
$$[1 - (1 - m) \, z] \cosh z \, k \, d \, z$$
$$= \frac{l \, r^2}{E \, I_c \, (g - 1)} \left\{ - \left(\frac{y_c}{r} + \frac{1}{g - 1} \right) \int_0^1 \cosh z \, k \, d \, z \right.$$
$$+ \frac{1}{g - 1} \int_0^1 \cosh^2 z \, k \, d \, z - (1 - m) \left[- \left(\frac{y_c}{r} + \frac{1}{g - 1} \right) \right.$$
$$\left. \left. \int_0^1 z \cosh z \, k \, d \, z + \frac{1}{g - 1} \int z \cosh^2 z \, k \, d \, z \right] \right\}$$
$$= \frac{l \, r^2}{E \, I_c \, (g - 1)} \left\{ - \left(\frac{y_c}{r} + \frac{1}{g - 1} \right) \cdot \frac{\sqrt{g^2 - 1}}{k} \right.$$
$$+ \frac{1}{g - 1} \left(\frac{g \sqrt{g^2 - 1}}{2 \, k} + \frac{1}{2} \right) - (1 - m)$$

$$\left[-\left(\frac{y_c}{r}+\frac{1}{g-1}\right)\left(\frac{\sqrt{g^2-1}}{k}-\frac{g-1}{k^2}\right)\right.$$
$$\left.+\frac{1}{g-1}\left(\frac{g\sqrt{g^2-1}}{2k}-\frac{g^2-1}{4k^2}+\frac{1}{4}\right)\right]\Bigg\}$$

or, in more convenient form,

$$\int_L^R y^2\,dw=\frac{l\,r^2}{E\,I_c}\left\{\left(\frac{g-2}{2(g-1)}-\frac{y_c}{r}\right)\frac{\sqrt{g^2-1}}{k(g-1)}+\frac{1}{2(g-1)^2}\right.$$
$$-(1-m)\left[\left(\frac{g-2}{2(g-1)}-\frac{y_c}{r}\right)\frac{\sqrt{g^2-1}}{k(g-1)}+\frac{1}{4(g-1)^2}\right.$$
$$\left.\left.-\frac{1}{k^2}\left(\frac{g-3}{4(g-1)}-\frac{y_c}{r}\right)\right]\right\}\qquad(63)$$

Table 18 gives the solution of Equation (63).

TABLE 18.—VALUE OF $\int y^2\,dw$ (BY EQUATION (63)).

$N.$	$g.$	VALUE OF $\frac{E\,I_c\int y^2\,dw}{l\,r^2}=C.$					
		$m=0.15.$	$m=0.20.$	$m=0.25.$	$m=0.30.$	$m=0.40.$	$m=0.50.$
0.25	1.000	0.0329	0.0370	0.0410	0.0448	0.0520	0.0588
0.24	1.347	0.0320	0.0361	0.0400	0.0437	0.0509	0.0577
0.23	1.756	0.0312	0.0352	0.0390	0.0428	0.0498	0.0566
0.22	2.240	0.0302	0.0342	0.0380	0.0417	0.0486	0.0553
0.21	2.814	0.0294	0.0332	0.0370	0.0407	0.0475	0.0541
0.20	3.500	0.0285	0.0323	0.0361	0.0396	0.0466	0.0530
0.19	4.324	0.0276	0.0314	0.0351	0.0386	0.0455	0.0519
0.18	5.321	0.0267	0.0305	0.0341	0.0376	0.0443	0.0507
0.17	6.536	0.0258	0.0295	0.0331	0.0366	0.0432	0.0496
0.16	8.031	0.0250	0.0286	0.0322	0.0356	0.0422	0.0484
0.15	9.889	0.0241	0.0277	0.0312	0.0346	0.0411	0.0472

The numerator, δ_{HA}, for a single unit load, P, is obtained by integrating the differential equation for the elastic line,

$$\frac{d^2\,\delta}{d\,z^2}=-\frac{M\,l_1^2}{E\,I_c}[1-(1-m)\,z]$$

For

$$H=1,\ M=M\,y=y=y_c-y_0$$

hence,

$$\frac{d^2\,\delta_{HA}}{d\,z^2}=-\frac{r\,l^2}{4\,E\,I_c}[1-(1-m)\,z]\left[\frac{y_c}{r}-\frac{1}{g-1}(\cosh z\,k-1)\right]$$

After integrating twice, introducing the proper constants of integration, this reduces to the following:

$$\delta_{HA}=\frac{r\,l^2}{4\,E\,I_c}\left\{\left(\frac{y_c}{r}+\frac{1}{g-1}\right)\frac{1-z^2}{2}-\frac{1}{k^2}\left(1-\frac{y_0}{r}\right)\right.$$
$$-(1-m)\left[\left(\frac{y_c}{r}+\frac{1}{g-1}\right)\frac{1-z^3}{6}-\frac{1}{k^2}\left(1-z\,\frac{y_0}{r}\right)\right.$$
$$\left.\left.-\frac{2}{k^2(g-1)}\left(1-z-\frac{\sqrt{g^2-1}}{k}+\frac{1}{k}\sqrt{\left\{1+\frac{y_0}{r}(g-1)\right\}^2-1}\right)\right]\right\}\qquad(64)$$

The value of u will be small for all but very flat arches and need not be considered ordinarily in calculating the live load stresses if the rib-shortening effect is considered separately:

$$u = \frac{\int_L^R \cos\phi \frac{dx}{EA}}{\int_L^R y^2\, dw} = \frac{l}{E A_m \int y^2\, dw}$$

The value of $\int y^2\, dw$ has just been derived and A_m can be obtained from Fig. 33.

Numerical Example.—The values of H, y_c, $\int y^2\, dw$, and u will now be determined for the arch rib used in the previous examples:

$l = 100$ ft. $= 1\,200$ in. $\quad m = 0.339$
$r = 15$ ft. $= 180$ in. $\quad g = 4.70$
$I_c = 6\,399$ in.^{4} $\quad k = \cosh^{-1} g = 2.2292$

$$y_c = \frac{2r}{(g-1)(1+m)}\left[\frac{\sqrt{g^2-1}}{k} - 1 - (1-m)\left(\frac{\sqrt{g^2-1}}{k} - \frac{g-1}{k^2} - \frac{1}{2}\right)\right]$$

$$= \frac{2 \times 15}{3.70 \times 1.339}\left[\frac{4.59}{2.229} - 1 - 0.661\left(\frac{4.59}{2.229} - \frac{3.70}{4.97} - \frac{1}{2}\right)\right] = 3.15 \text{ ft.} = 37.8 \text{ in.}$$

The value obtained graphically was 3.14 ft.

$$\int y^2\, dw = \frac{l r^2}{E I_c}\left\{\left(\frac{g-2}{2(g-1)} - \frac{y_c}{r}\right)\frac{\sqrt{g^2-1}}{k(g-1)} + \frac{1}{2(g-1)^2}\right.$$
$$- (1-m)\left[\left(\frac{g-2}{2(g-1)} - \frac{y_c}{r}\right)\frac{\sqrt{g^2-1}}{k(g-1)} + \frac{1}{4(g-1)^2}\right.$$
$$\left.\left. - \frac{1}{k^2}\left(\frac{g-3}{4(g-1)} - \frac{y_c}{r}\right)\right]\right\}$$

All dimensions will be expressed in inches.

$$\int y^2\, dw = \frac{1\,200 \times 180 \times 180}{2\,000\,000 \times 6\,399}\left\{\left(\frac{2.70}{7.40} - \frac{37.8}{180}\right)\frac{4.59}{2.229 \times 3.70} + \frac{1}{2 \times 13.68}\right.$$
$$- 0.661\left[\left(\frac{2.70}{7.40} - \frac{37.8}{180}\right)\frac{4.59}{2.229 \times 3.70} + \frac{1}{4 \times 13.68}\right.$$
$$\left.\left. - \frac{1}{2.229 \times 2.229}\left(\frac{1.70}{14.80} - \frac{37.8}{180}\right)\right]\right\} = 0.0001246$$

This compares with the value obtained graphically, 0.0001223.

$$\delta_{HA} = \frac{r\, l^2}{4 E I_c}\left\{\left(\frac{y_c}{r} + \frac{1}{g-1}\right)\frac{1-z^2}{2} - \frac{1}{k^2}\left(1 - \frac{y_0}{r}\right)\right.$$
$$- (1-m)\left[\left(\frac{y_c}{r} + \frac{1}{g-1}\right)\frac{1-z^3}{6} - \frac{1}{k^2}\left(1 - z\frac{y_0}{r}\right)\right.$$
$$\left.\left. - \frac{2}{k^2(g-1)}\left(1 - z - \frac{\sqrt{g^2-1}}{k} + \frac{1}{k}\sqrt{\left\{1 + \frac{y_0}{r}(g-1)\right\}^2 - 1}\right)\right]\right\}$$

Solving for δ_{HA} for a load at the center of the span with $z = 0$ and $y_0 = 0$:

$$\delta_{HA} = \frac{180 \times 1\,440\,000}{4 \times 2\,000\,000 \times 6\,399}\left\{\left(\frac{37.8}{180} + \frac{1}{3.70}\right)\frac{1}{2} - \frac{1}{2.229 \times 2.229}\right.$$
$$- 0.661\left[\left(\frac{37.8}{180} + \frac{1}{3.70}\right)\frac{1}{6} - \frac{1}{2.229 \times 2.229}\right.$$
$$\left.\left. - \frac{2}{2.229 \times 2.229 \times 3.70}\left(1 - 0 - \frac{4.59}{2.229} + 0\right)\right]\right\} = 0.0002175$$

Then, H for a unit load at the center of the span is:

$$H = \frac{\delta_{HA}}{\delta_{HL}} = \frac{0.0002175}{0.0001246} = 1.748$$

The value of H determined graphically is 1.752.

The value of u can now be computed; from Fig. 33,

$$A_m = 1.230 \times A_c = 1.23 \times 18 \times 12 = 265.6 \text{ sq. in.}$$

$\int y^2\, d\,w$ as previously computed is 0.0001246. Then,

$$u = \frac{l}{E\, A_m \int y^2\, d\,w} = \frac{1\,200}{2\,000\,000 \times 265.6 \times 0.0001246} = 0.01813$$

From the graphical solution, u was computed to be 0.0185.

The values of the horizontal and vertical reactions due to the dead load are given by Table 9, as well as the values of tan ϕ_s. Figs. 28 to 32 are influence lines for the horizontal thrust due to the live load.

Temperature and Shrinkage Effects.—For the thrust due to a change in temperature, Equation (24*a*) gives:

$$H_t = \frac{t\,\alpha\,l}{\int_L^R y^2\, d\,w}.$$

Table 18 gives the values of

$$C = \frac{E\, I_c}{l\, r^2} \int y^2\, d\,w;$$

then

$$\int y^2\, d\,w = C\,\frac{l\, r^2}{E\, I_c}$$

and

$$H_t = \frac{t\,\alpha\, E\, I_c}{C\, r^2}$$

For extremely flat arches, this value should be multiplied by $\frac{1}{1+u}$.

The value of y can be computed by means of y_c taken from Table 17. The moment due to temperature change will be:

$$M_t = - H_t\, y$$

Fig. 56 has been prepared to facilitate the computation of temperature thrust. It gives the value of a coefficient, C_t, which equals $\frac{H_t\, r^2}{t\, I_c}$, or,

$$H_t = C_t\,\frac{t\, I_c}{r^2}; \text{ and } C_t = \frac{\alpha\, E}{C}$$

Fig. 56 is constructed with the following values: $\alpha = 0.000006$, and $E = 2\ 000\ 000$, with r, in feet, and I_c, in inches[4].

The effect of shrinkage of the concrete can be computed as the effect of an equivalent drop in temperature.

Rib-Shortening Effect.—The rib-shortening effect can be expressed as in Equation (21*a*) by:

$$H_{RS} = -H\frac{u'}{1+u} = \text{approximately} -H\,u'$$

in which, H is the horizontal thrust produced by dead and live loads and,

$$u' = \frac{\int \frac{d\,x}{E\,A\cos\phi}}{\int y^2\,d\,w} = \frac{l}{E\,A_m'\int y^2\,d\,w} = \frac{I_c}{A\,C_m'\,C\,r^2}$$

The value of C can be taken from Table 18 and C_m' from Fig. 33. I_c is the moment of inertia and A the area of the cross-section of the rib at the crown. The moments due to rib-shortening are:

$$M_{RS} = -H_{RS}\,y$$

Influence Lines

For convenience, the influence lines facilitating the design of concrete arches, Figs. 28 to 56, inclusive, are grouped together at the end of the paper. Figs. 28 to 32 and 34 to 49 give the influence lines for the horizontal thrust, vertical reaction, and for the bending moments at the crown, quarter-points, and springing line, for a range of values, m and N, covering all conditions in actual practice from a flat open spandrel type to a filled arch of high rise.

These curves give the values obtained from the solution of the formulas already derived. They may be used for determining the stresses produced by concentrated loads, and for computing the equivalent uniformly distributed loads to use in connection with the coefficients given in Figs. 50 to 55. The bending-moment influence lines show exactly where the loads should be placed for maximum moments.

Numerical examples will be given of the use of these diagrams for an arch of the following dimensions:

$$l = 100 \text{ ft.} \qquad N = 0.18604$$

$$r = 15 \text{ ft.} \qquad g = \frac{w_s}{w_c} = 4.70$$

$$m = \frac{I_c}{I_s\cos\phi_s} = 0.339$$

Horizontal Thrust.—Figs. 28 to 32 give the horizontal thrust computed by Equations (63) and (64). If the rib-shortening effect is computed separately, the term, u, can be neglected and the value of H becomes $P\,C_H\frac{l}{r}$.

Example.—To find the horizontal thrust produced by a load of 100 lb. at the center of the arch rib:

From Fig. 31, for $N = 0.186 \ldots . C_H = 0.2650$
From Fig. 32, for $N = 0.186 \ldots . C_H = 0.2590$
Interpolating for $m = 0.339 \ldots . C_H = 0.2630$

Then,

$$H = 0.2630 \times 100 \times \frac{100}{15} = 175.3 \text{ lb.}$$

Vertical Reaction.—Table 16 gives the vertical reaction computed by Equation (58) for single concentrated loads. Fig. 34 has been prepared for finding the vertical reaction under uniformly distributed loading over part of the span.

Example 1.—To find the vertical reaction at the left end produced by a load of 100 lb. at the quarter-point, 5:

From Table 16, for $m = 0.339 \ldots . V_L = 100 \times 0.864 = 86.4$ lb.

Example 2.—To find the vertical thrust at the left end produced by a load of 100 lb. per lin. ft. from Point 5 to Point 10:

From Fig. 34, for a load from the left end to Point 10 $\ldots . \frac{V_L}{w\,l} = 0.414$

From Fig. 34, for a load from the left end to Point 5 $\ldots . \frac{V_L}{w\,l} = 0.239$

From Fig. 34, for a load from Point 5 to Point 10 $\ldots . \frac{V_L}{w\,l} = 0.175$

Then,

$$V_L = 0.175 \times 100 \times 100 = 1\ 750 \text{ lb.}$$

Bending Moments.—Figs. 35 to 49 give the values of the bending moments at the crown, springing line, and quarter-points, produced by a single unit load at any position on the span. These are computed from the values of M', V_L, and H.

Example.—To find the bending moment at the springing produced by a load of 100 lb. at the center of the span:

From Fig. 43, for $m = 0.30$ and $N = 0.186 \ldots . \frac{M}{P\,l} = 0.0636$

From Fig. 44, for $m = 0.40$ and $N = 0.186 \ldots . \frac{M}{P\,l} = 0.0590$

Interpolating for $m = 0.339 \ldots . \frac{M}{P\,l} = 0.0620$

Then,

$$M = 100 \times 100 \times 0.0620 = 620 \text{ ft-lb.}$$

Tables and Diagrams for Arch Rib Design

In designing, Tables 6, 7, 9, 12, 13, and 15 to 18, inclusive, and Figs. 28 to 56, inclusive, may be used instead of the equations, as they give values directly for the design or investigation of arch ribs. After the value of $g = \frac{w_s}{w_c}$ has been computed, the value of N, H_d V_d, and $\tan \phi_s$ can be obtained from Table 9.

Table 17 gives y_c, the vertical distance from the arch axis at the center to the elastic center of gravity of the rib through which the horizontal thrusts are assumed to act. This distance is used for computing the moments due to rib-shortening and the temperature change after the thrust has been determined.

The values in Table 18 give $\int y^2\, d\,w$ when multiplied by $\frac{l\,r^2}{E\,I_c}$. This is used for computing the value of u:

$$u = \frac{\int \cos\phi \frac{d\,x}{E\,A}}{\int y^2\, d\,w} = \frac{l}{E\,A_m \int y^2\, d\,w}$$

The values in Table 18 are:

$$\frac{E\,I_c \int y^2\, d\,w}{l\,r^2} = C$$

hence,

$$\int y^2\, d\,w = C\,\frac{l\,r^2}{E\,I_c}$$

and,

$$u = \frac{l}{E\,A_m\,C\,\frac{l\,r^2}{E\,I_c}} = \frac{I_c}{A_m\,C\,r^2} = \frac{I_c}{A\,C_m\,C\,r^2}$$

in which, $C_m = \frac{A_m}{A}$, which is given by Fig. 33.

Similarly,

$$u' = \frac{\int \frac{d\,x}{E\,A\cos\phi}}{\int y^2\, d\,w} = \frac{I_c}{A\,C'_m\,C\,r^2}$$

The term, A, is the area of the cross-section of the rib at the crown. It must be kept in mind that in solving the equations all those terms involving units of length, as I_c, A, r, l, etc., must be expressed either in inches or in feet.

Live Load Moments and Thrusts.—Figs. 50 to 55 give the maximum positive and negative bending moments in the crown, springing line, and quarter-point sections, together with the corresponding values of the horizontal live load thrust produced by the loading that gives maximum moments. The moments given by the diagrams are those produced by a uniform load of p lb. per lin. ft. of rib so placed on the span as to produce the maximum moment. The sum of the maximum positive and negative moments will be the moment produced by a full uniformly distributed live load.

The exact position of the loading can be found from the moment influence lines. When the position of the loading is known, the corresponding vertical reaction can be obtained from Fig. 34.

Example.—Assume:

$l = 100$ ft. $\qquad r = 20$ ft.
$N = 0.18 \qquad m = 0.30$
$p = 200$ lb. per lin. ft.

From Fig. 53:

Max. negative, M, at springing line $= 0.01675 \times 200 \times 100^2 = -33\ 500$ ft-lb.

$$\text{Corresponding } H = 0.0336 \times 200 \times \frac{100^2}{20} = 3\ 360 \text{ lb.}$$

From Fig. 52:

Max. positive, M, at springing line $= 0.0308 \times 200 \times 100^2 = +61\ 600$ ft-lb.

$$\text{Corresponding } H = 0.1025 \times 200 \times \frac{100^2}{20} = 10\ 250 \text{ lb.}$$

For full load:

$$M = 61\ 600 - 33\ 500 = +28\ 100 \text{ ft-lb.}$$
$$H = 10\ 250 + 3\ 630 = 13\ 880 \text{ lb.}$$
$$V = 200 \times \frac{100}{2} = 10\ 000 \text{ lb.}$$

Fig. 43 shows that for the maximum negative, M, at the left springing line, the load should extend from the left end for 0.722 of the distance to the center. Fig. 34 gives for the vertical reaction at the left end due to this loading:

$$V_L = 0.328 \times 200 \times 100 = 6\ 560 \text{ lb.}$$

Then, for the maximum positive moment:

$$V_L = 10\ 000 - 6\ 560 = 3\ 440 \text{ lb.}$$

In order that the bending moment influence lines may be easily constructed for any section of the rib, the values of M' have been computed and are given in Table 15. The values of V_L may be taken from Table 16 and H_L from Figs. 28 to 32. The bending moment at any point, O, produced by a unit load is given by Equation (33):

$$M_0 = M' + V_L x - H_L y + M_p$$

in which, M_p is the moment produced at O by the superimposed load when the rib is acting as a cantilever with the right end fixed. The origin of the coordinates, x and y, is of course at the elastic center of gravity of the rib.

Example.—To find the bending moment produced in the rib of the examples of Sections IV and V at Point 2 by a unit load at the center: $l = 100$ ft., and $r = 15$ ft. At Point 2, $y_0 = 8.35$ ft.; as y_c was found to be 3.15 ft., $y = 3.15 - 8.35 = -5.20$ ft. Also, at Point 2, $x = -40$ ft. and $M_p = 0$. Previously it was found that $H_L = 1.748$ and $V_L = 0.500$ for a unit load at the center.

From Table 15, for $m = 0.339$ with the load at Point 10:

$$M' = 0.10442 \times 100 = 10.442 \text{ ft-lb.}$$

then,

$$M_0 = 10.442 + (-40 \times 0.50) - (-5.20 \times 1.748) - 0$$
$$= -0.4684 \text{ ft-lb.}$$

Thickness of Arch Rib.—When it is desired to design an arch rib for fixed conditions of span, rise, and loading, by first assuming a design of the rib the stresses can be quickly computed by the tables and diagrams, and the proper thickness of rib determined.

The thrust and moment due to the dead and live loads, temperature, and rib-shortening, can be expressed as follows:

$$T = T_a + T_l + T_t + T_{RS} = \sec \phi \, (H_a + H_l + H_t + H_{RS}) \text{ (approximately)}$$
$$= \sec \phi \left[C_a \frac{w_c \, l^2}{r} + C_l \frac{p \, l^2}{r} + C_t \frac{t \, I_c}{r} - (H_a + H_l) \, u' \right] \text{ (approximately)}$$

or,

$$T = \sec \phi \left[\frac{l^2}{r} (C_a \, w_c + C_l \, p) \, (1 - u') + C_t \frac{t \, I_c}{r} \right] \text{ (approximately)}$$

Also,

$$M = M_l + M_t + M_{RS}$$
$$= K_l \, p \, l^2 + H_t \, y + (H_a + H_l) \, u' \, y$$

This does not include the moment due to dead load concentrations which must be considered in the final design of an open spandrel arch.

When the values of T and M are determined, it is necessary to compute the thickness of rib required. For any given allowable maximum fiber stress this thickness will depend on the amount of reinforcement used. For fixed values of T and M, it is more economical to use a thick rib and a low percentage of reinforcement; however, an increase in the rib thickness increases both the dead load thrust and the temperature bending. It has been found that a satisfactory value for the thickness will usually be obtained for ordinary cases if the rib is calculated as a homogeneous section without reinforcement. This may be done as follows:

$$f_c = \frac{T}{b \, d} \pm \frac{6 \, M}{b \, d^2}$$

in which, f_c is the allowable fiber stress in the concrete. Then, for a rib 12 in. wide ($b = 12$) and for M, in foot-pounds:

$$f_c = \frac{T}{12 \, d} \pm \frac{M}{2 \, d^2} = \frac{T}{12 \, d} \pm \frac{6 \, M}{d^2}$$

Solving for d,

$$d = \frac{T}{24 \, f_c} + \sqrt{\frac{T^2}{576 \, f_c^2} + \frac{6 \, M}{f_c}}$$

in which, d is in inches and M in foot-pounds.

When the thickness of the rib is determined by this formula the amount of steel required will vary with the eccentricity of the thrust. It will be found that when this eccentricity is small, the amount of steel required may be less than it is desirable to use. In that case, a depth of rib less than that given by the formula may be used to advantage.

For flat arches, in which the temperature stresses are high, there may be some advantage in using a thinner rib with more steel. As the temperature moments increase with the moment of inertia of the section while the allowable moment increases with the section modulus, the temperature stresses can be reduced if necessary by using a thinner rib with more steel.

After the thickness of the rib and the actual reinforcement have been selected, the stresses in the concrete and steel should be computed by the usual method.

Determination of the Best Proportions for an Arch Rib.—By means of the tables and diagrams of this paper, a much more complete investigation of the

effect of varying proportions on the stresses in the arch rib can be made than would be practical with ordinary methods. An example of this is shown in Fig. 27 which was computed for a 100-ft. span with 15-ft. rise. The left-hand side of the diagram shows the effect of varying crown thicknesses on the stresses in the concrete at the crown. For this determination it was necessary to assume a value of the ratio of d_s to d_c. This is usually between 1.5 and 2.0, the higher value being suitable for flat arches. Too great a thickness at the springing line should be avoided as it increases the temperature and rib-shortening stresses.

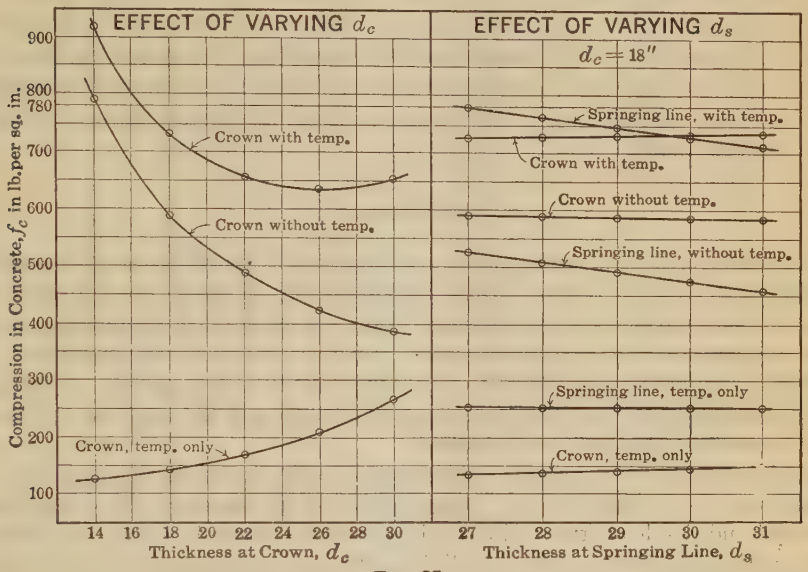


FIG. 27.

The thickness at the crown should be established first as it has more effect on the stresses at the springing line than the thickness at the springing line has on the crown stresses. One particularly interesting fact shown by the crown-stress curve of Fig. 27 is that there is a certain thickness which gives a minimum combined stress, beyond which an increase in thickness causes the temperature and rib-shortening stresses to increase more rapidly than the dead and live load stresses are decreased.

After the crown thickness has been determined, similar curves can be computed for the stress at the springing line and the crown for different springing line thicknesses, as shown in the right-hand side of Fig. 27. The correct values of m and N should be used for each different springing line thickness. It is seen that by increasing the springing line thickness and keeping the crown thickness constant, the springing line stress decreases, but the crown stress increases slightly. By means of such curves it is possible to determine exactly the proper thickness of rib at both springing line and crown for any desired or specified values of allowable stresses.

The tables have been computed by the writer, but the majority of the influence line diagrams giving moments and thrusts have been prepared with the assistance of tables given by Strassner* which have greatly decreased the labor that would otherwise have been required in computing all the quantities.

Comparison of Two Methods.—In order to show how closely the results given by the formulas developed in this paper check the results of graphical analysis, there have been given in Table 19 the ordinates of the arch axis and the live load thrust and moment influence line ordinates as determined for the 100-ft. span treated in Section III. The two methods agree within the limits of the accuracy of slide-rule computations. Table 20 gives the dead load moments for the rib, the axis of which was determined by Equation (38). The total dead load moments at the crown and the springing line are so small that they may be neglected when the load is distributed as in the case of a filled spandrel arch. The effect of concentrated dead loads, of course, must be considered.

Conclusion

The data presented herein apply quite generally to all types of hingeless symmetrical concrete arches, either open-spandrel or filled, of any span or rise, with solid barrels or separate ribs. They can be applied to unsymmetrical ribs, as mentioned in the Introduction. In case architectural appearance or limited clearance prevents the use of a form for the axis as given in Section IV, the method based on that form cannot be used and the analysis can then best be made by the graphical method.

The influence lines apply to the form of arch to which they correspond regardless of the dead loading. They can be used, therefore, for computing the dead and live load stresses in existing designs if there is not a great difference in the shape of the rib. A slight difference will have very little effect on the moments and thrusts.

The use of the form of rib defined by the equations of Sections IV and V makes possible a simpler analytical solution of the stresses in arches on elastic supports than would otherwise be possible as the elastic behavior of the separate arch ribs can easily be determined and equations can be written for the movements at the supports.

As a further development, the tables and diagrams could be used to establish data on the proper size of rib for arches of different span and rise, provided that a definite specification for loading, temperature variation, allowable stresses, etc., were adopted. The ratio of dead to live load is an important factor and so is the shrinkage of the concrete. There are so many variables which greatly effect the rib design that no accurate general expression for rib thickness can be derived.

The writer believes that no methods as simple as those given herein can be developed for the treatment of hingeless symmetrical arches, which will be as accurate or have as general an application.

* "Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogenträger."

TABLE 19.—Comparison of Arch Design by Algebraic and by Analytical Methods.

Point.	y_0		H		V		M_o		M_s	
	Analytically.	By Equation (38).	Graphically.	By Diagram.	Graphically.	By Diagram.	Graphically.	By Diagram.	Graphically.	By Diagram.
0	15.00	**15.00**	0.000	**0.000**	1.000	**1.000**	−0.00	**−0.00**	−0.00	**−0.00**
1	11.31	**11.30**	0.049	**0.052**	0.994	**0.995**	−0.05	**−0.09**	−4.02	**−4.03**
2	8.38	**8.35**	0.189	**0.191**	0.979	**0.978**	−0.34	**−0.31**	−6.46	**−6.38**
3	6.06	**6.03**	0.396	**0.398**	0.952	**0.951**	−0.58	**−0.57**	−7.24	**−7.18**
4	4.23	**4.20**	0.643	**0.648**	0.914	**0.913**	−0.80	**−0.77**	−6.86	**−6.72**
5	2.81	**2.79**	0.912	**0.916**	0.864	**0.864**	−0.81	**−0.79**	−5.34	**−5.27**
6	1.73	**1.72**	1.174	**1.174**	0.806	**0.805**	−0.54	**−0.53**	−3.22	**−3.18**
7	0.95	**0.94**	1.407	**1.406**	0.738	**0.737**	+0.08	**+0.09**	−0.72	**−0.71**
8	0.41	**0.41**	1.592	**1.590**	0.664	**0.663**	+1.14	**+1.16**	+1.82	**+1.87**
9	0.10	**0.10**	1.712	**1.710**	0.583	**0.582**	+2.75	**+2.75**	+4.29	**+4.21**
10	0.00	**0 00**	1.752	**1.751**	0.500	**0.500**	+4.93	**+4.92**	+6.21	**+6.19**
9′	0.10	**0.10**	1.712	**1.710**	0.417	**0.418**	+2.75	**+2.75**	+7.59	**+7.50**
8′	0.41	**0.41**	1.592	**1.590**	0.336	**0.337**	+1.15	**+1.16**	+8.23	**+8.14**
7′	0.95	**0.94**	1.407	**1.406**	0.262	**0.263**	+0.08	**+0.09**	+8.08	**+8.07**
6′	1.73	**1.72**	1.174	**1.174**	0.194	**0.195**	−0.54	**−0.53**	+7.38	**+7.38**
5′	2.81	**2.79**	0.912	**0.916**	0.136	**0.136**	−0.78	**−0.79**	+6.09	**+6.15**
4′	4.23	**4.20**	0.643	**0.648**	0.086	**0.087**	−0.77	**−0.77**	+4.57	**+4.61**
3′	6.06	**6.03**	0.396	**0.398**	0.048	**0.049**	−0.57	**−0.57**	+2.97	**+2.99**
2′	8.38	**8.35**	0.189	**0.191**	0.021	**0.022**	−0.30	**−0.31**	+1.48	**+1.52**
1′	11.31	**11.30**	0.049	**0.052**	0.006	**0 005**	−0.07	**−0.09**	+0.36	**+0.44**
0′	15.00	**15.00**	0.000	**0.000**	0.000	**0.000**	−0.00	**−0.00**	+0.00	**+0.00**
$\Sigma + M$							+12.88	**+12.92**	+59.07	**+59.07**
$\Sigma - M$							− 6.15	**− 6.12**	−33.86	**−33.47**

TABLE 20.—Dead Load Moment at Crown and Springing Line.

Section	Dead load, W.	Crown.		Springing.	
		$\frac{M}{Wl}$	M, in foot-pounds.	$\frac{M}{Wl}$	M, in foot-pounds.
1	10 702	−0.00019	− 203	−0.0223	−23 900
2	8 595	−0.00190	−1 630	−0.0539	−46 300
3	6 915	−0.00450	−3 110	−0.0695	− 48 000
4	5 668	−0.00685	−3 880	−0.0707	−40 100
5	4 677	−0.00805	− 3 770	−0.0616	−28 800
6	3 925	−0.00694	−2 725	−0.0426	−16 700
7	3 361	−0.00270	− 908	− 0.0196	− 6 590
8	2 971	+0.00560	+1 664	+0.0056	+ 1 664
9	2 708	+0.01870	+5 074	+0.0296	+ 8 020
10	2 566	+0.03770	+9 680	+0.0528	+13 560
10′	2 566	+0.03770	+9 680	+0.0694	+17 710
9′	2 708	+0.01870	+5 074	+0.0792	+21 440
8′	2 971	+0.00560	+1 664	+0.0819	+24 330
7′	3 361	−0.00270	− 908	+0.0778	+26 150
6′	3 925	−0.00694	−2 725	+0.0681	+26 700
5′	4 677	−0.00805	−3 770	+0.0541	+25 300
4′	5 668	−0.00685	−3 880	+0.0380	+21 530
3′	6 915	−0.00450	−3 110	+0.0225	+15 560
2′	8 595	−0.00190	−1 630	+0.0090	+ 7 740
1′	10 702	−0.00019	− 203	+0.0011	+ 1 180
.....	.....	.....	$\Sigma M = +376$	.....	$\Sigma M = +494$

Summary of Notation

l = theoretical span of neutral axis of rib.

r = rise of neutral axis.

d_s = depth of rib at springing line.

d_c = depth of rib at crown.

x and y are the co-ordinates of the neutral axis measured from the vertical and horizontal lines through the elastic center of the rib.

$$z = \frac{2x}{l}$$

y_c = value of y at the crown.

N = the vertical distance from the rib axis at the quarter-point of the span to the point at the crown divided by the rise, r.

ϕ = the angle between the horizontal and the tangent to the neutral axis at any point.

ϕ_s = the value of ϕ at the springing line.

I_s = the moment of inertia of the rib at the springing line.

I_c = the moment of inertia of the rib at the crown.

$$m = \frac{I_c}{I_s \cos \phi_s}$$

A = the area of the cross-section of the rib.

$$A_m = \frac{l}{\int_L^R \frac{\cos \phi \, d x}{A}} = \text{the reciprocal of the mean value of } \frac{\cos \phi}{A}.$$

$$A'_m = \frac{l}{\int_L^R \frac{d x}{A \cos \phi}} = \text{the reciprocal of the mean value of } \frac{1}{A \cos \phi}.$$

E = the modulus of elasticity of the concrete.

E_s = the modulus of elasticity of the steel.

$$n = \frac{E_s}{E}.$$

$$d w = \text{the elastic weight of a short section of rib} = \frac{d s}{E I}$$

$$u = \frac{l}{E A_m \int y^2 \, d w}.$$

$$u' = \frac{l}{E A_m' \int y^2 \, d w}.$$

P = concentrated load.

p = uniform live load per linear foot of span.

p = pole length in graphical analysis.

w_u = load per unit length of beam.

w_s = dead load per linear foot at springing line.

w_c = dead load per linear foot at crown.

$g = \frac{w_s}{w_c}$.

$k = \cosh^{-1} g = \log \left(g + \sqrt{g^2 - 1}\right)$

t = change in temperature, in degrees.

α = coefficient of thermal expansion.

M = bending moment.

H = horizontal reaction.

V = vertical reaction.

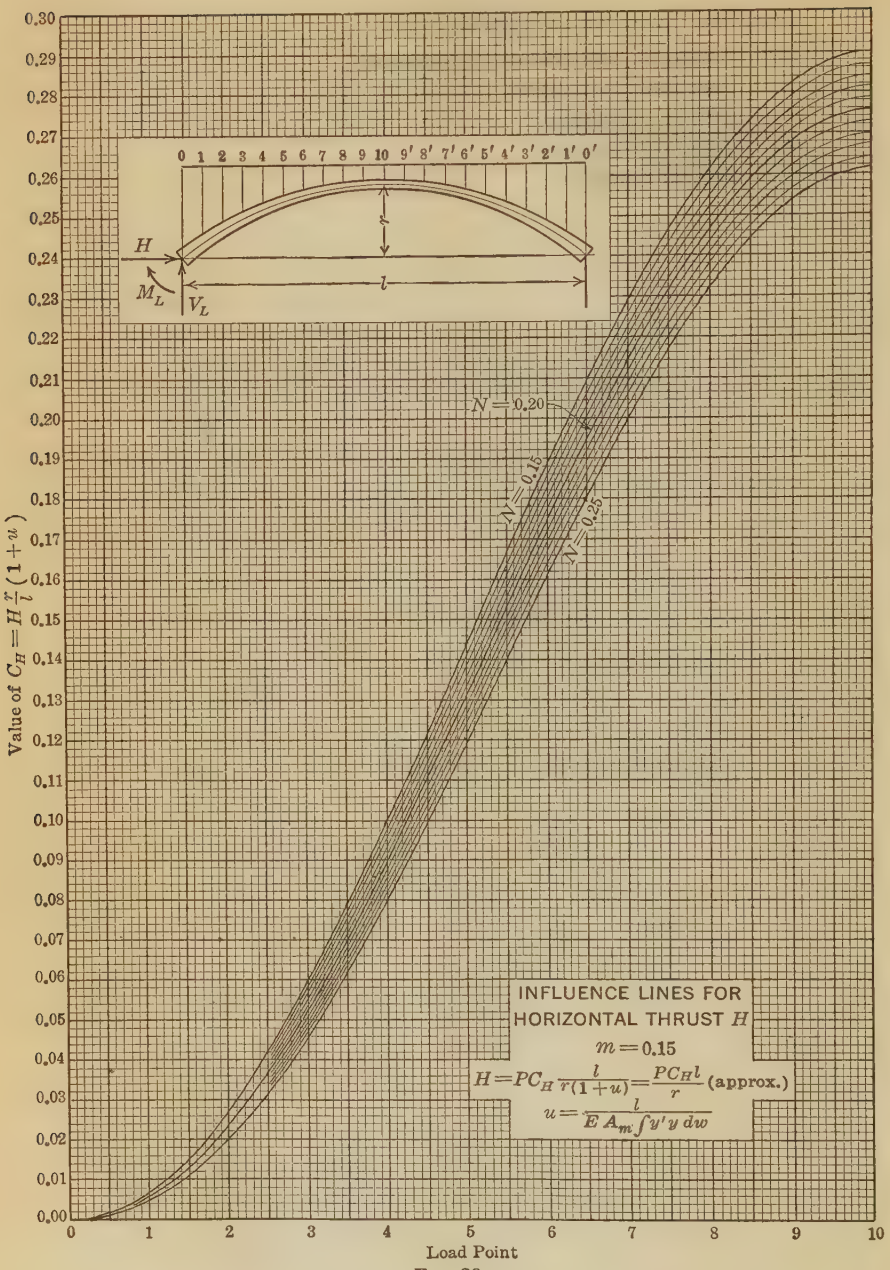
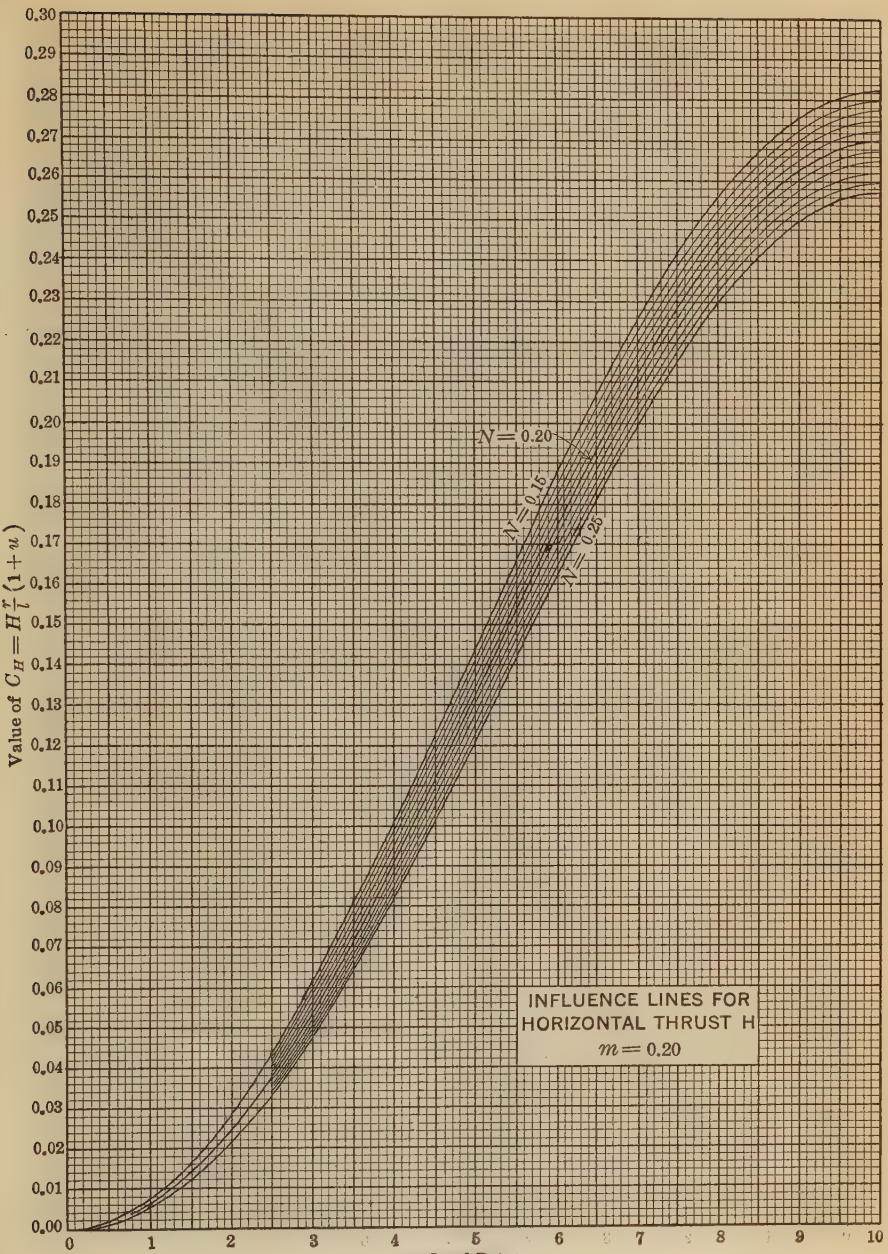

0 1 2 3 4 5 6 7 8 9 10 9′ 8′ 7′ 6′ 5′ 4′ 3′ 2′ 1′ 0′
H
M_L
V_L
l
r
N = 0.20
N = 0.15
N = 0.25
Value of $C_H = H\frac{r}{l}(1+u)$
Load Point
INFLUENCE LINES FOR HORIZONTAL THRUST H
m = 0.15
$H = PC_H \frac{l}{r(1+u)} = \frac{PC_H l}{r}$ (approx.)
$u = \frac{l}{E A_m \int y' y \, dw}$

FIG. 28.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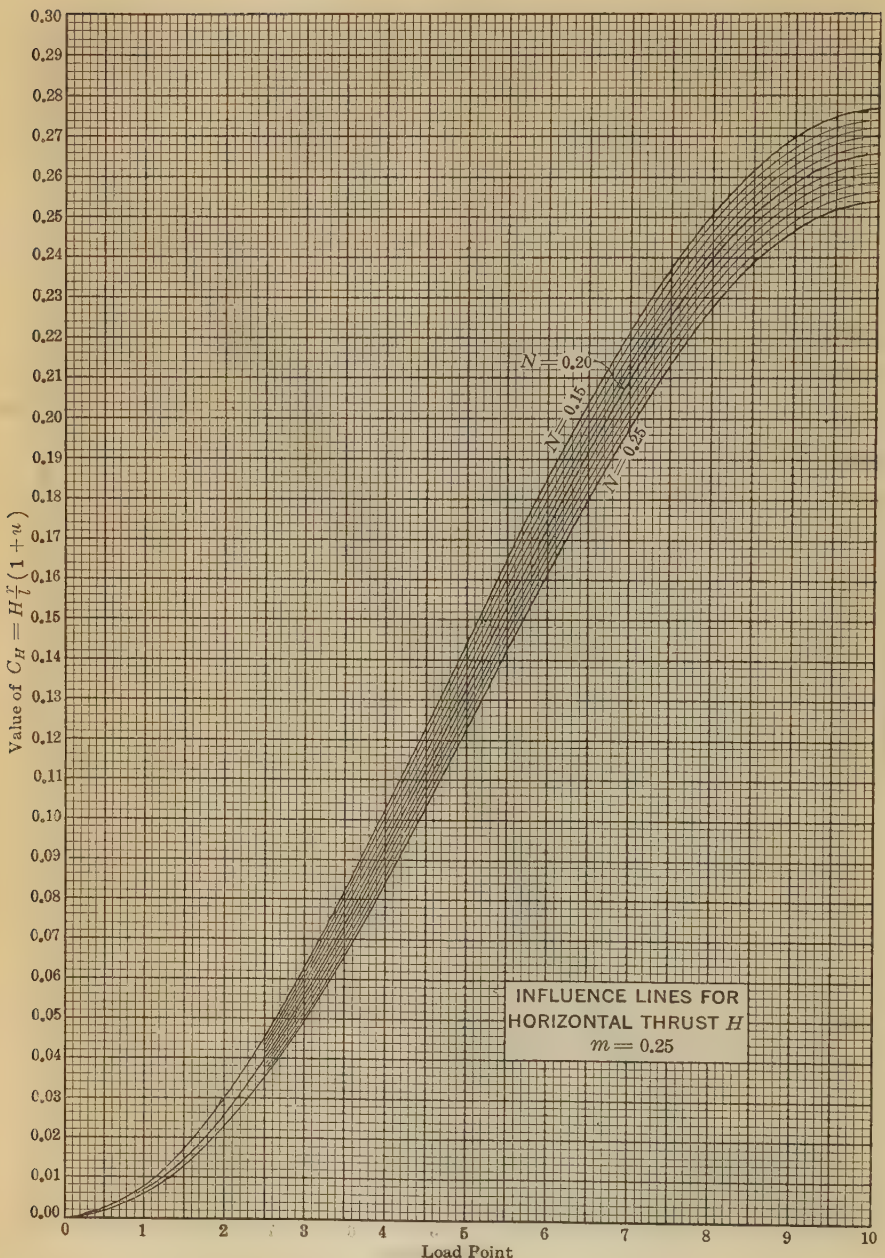
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
Value of $C_H = H\frac{r}{l}(1+u)$
$N = 0.20$
$N = 0.15$
$N = 0.25$
INFLUENCE LINES FOR
HORIZONTAL THRUST H
$m = 0.20$
0
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 29.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0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
Value of $C_H = H\frac{r}{l}(1+u)$
$N = 0.20$
$N = 0.15$
$N = 0.25$
INFLUENCE LINES FOR
HORIZONTAL THRUST H
$m = 0.25$
0
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 30.

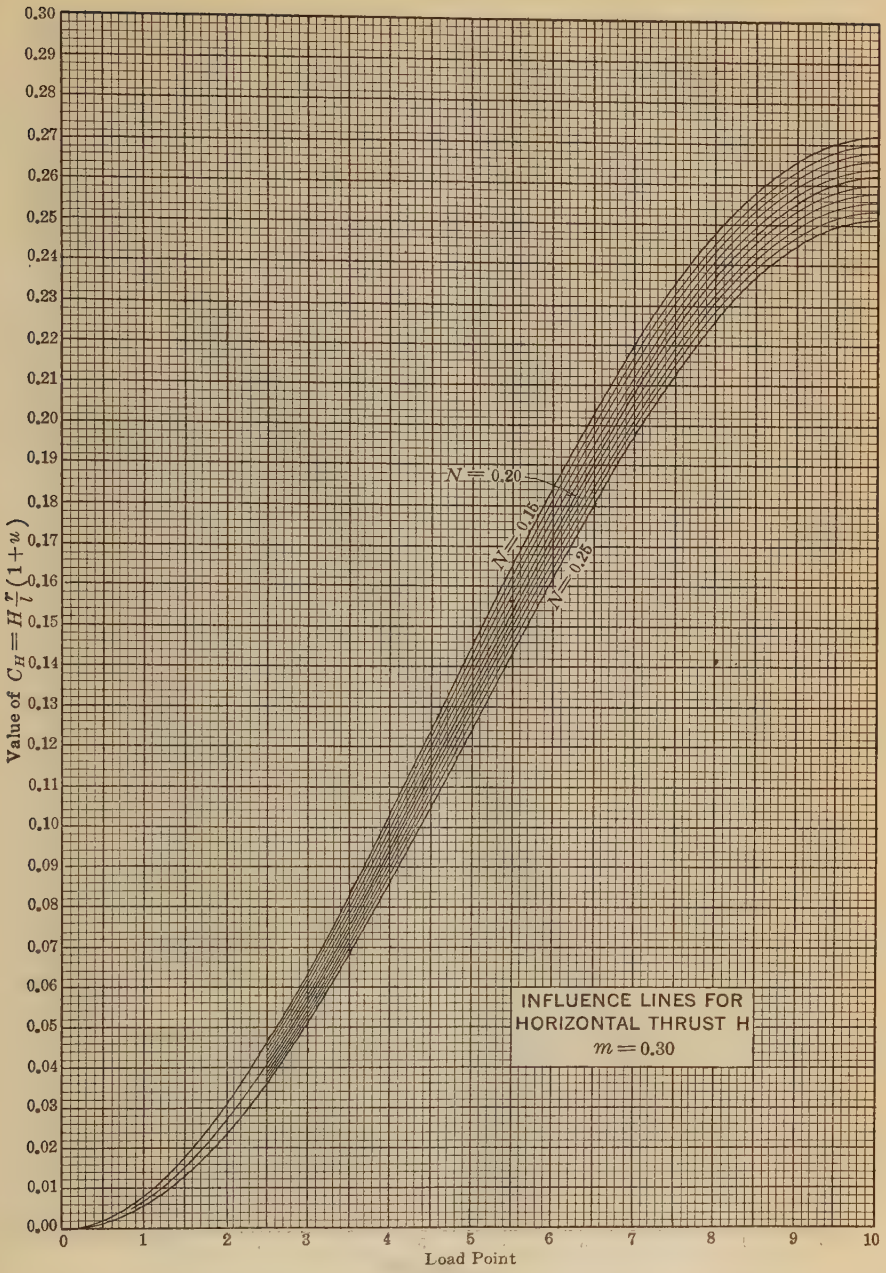
Value of $C_H = H\frac{r}{l}(1+u)$
0.30
0.29
0.28
0.27
0.26
0.25
0.24
0.23
0.22
0.21
0.20
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
0
1
2
3
4
5
6
7
8
9
10
Load Point
$N = 0.20$
$N = 0.15$
$N = 0.25$
INFLUENCE LINES FOR
HORIZONTAL THRUST H
$m = 0.30$

FIG. 31.

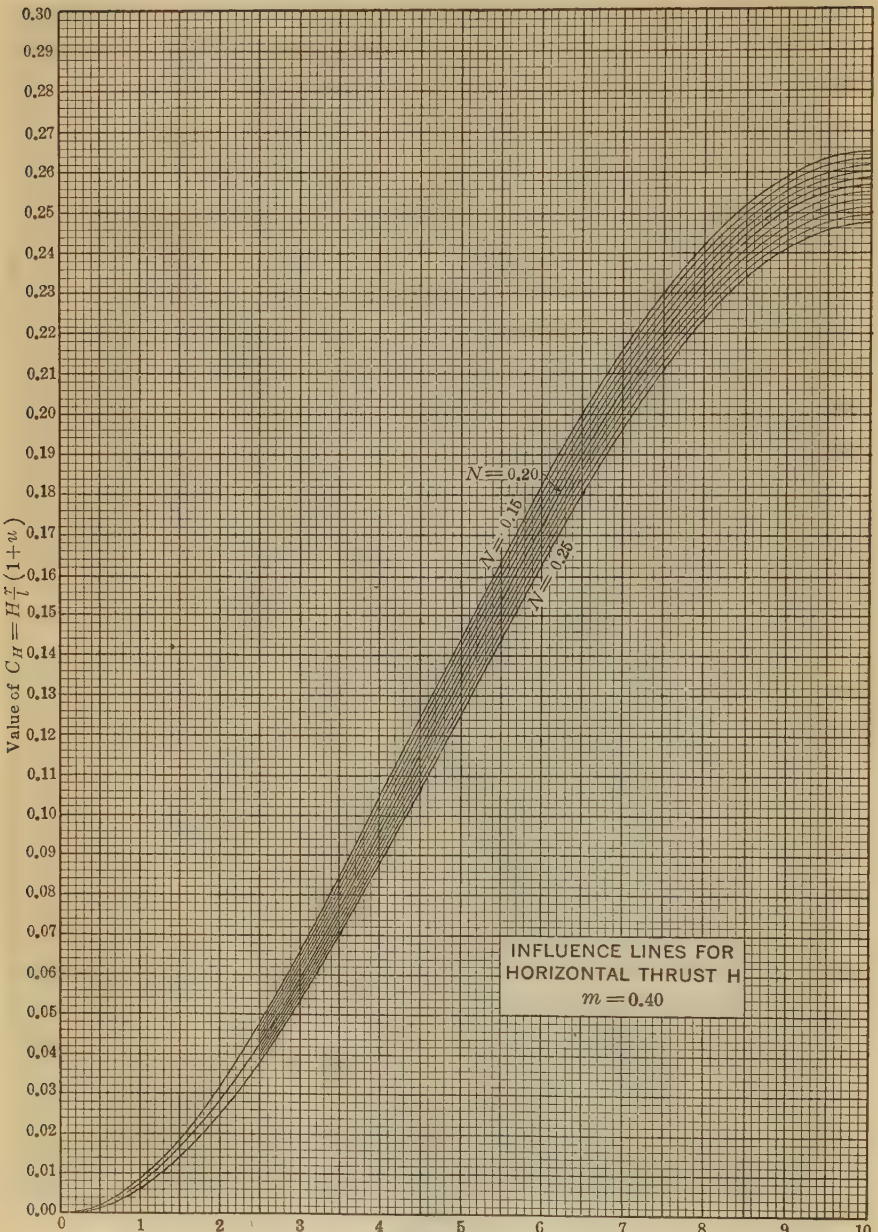
0.30
0.29
0.28
0.27
0.26
0.25
0.24
0.23
0.22
0.21
0.20
0.19
0.18
0.17
0.16
0.15
0.14
0.13
0.12
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
Value of $C_H = H\frac{r}{l}(1+u)$
N = 0.20
N = 0.15
N = 0.25
INFLUENCE LINES FOR
HORIZONTAL THRUST H
m = 0.40
0
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 32.

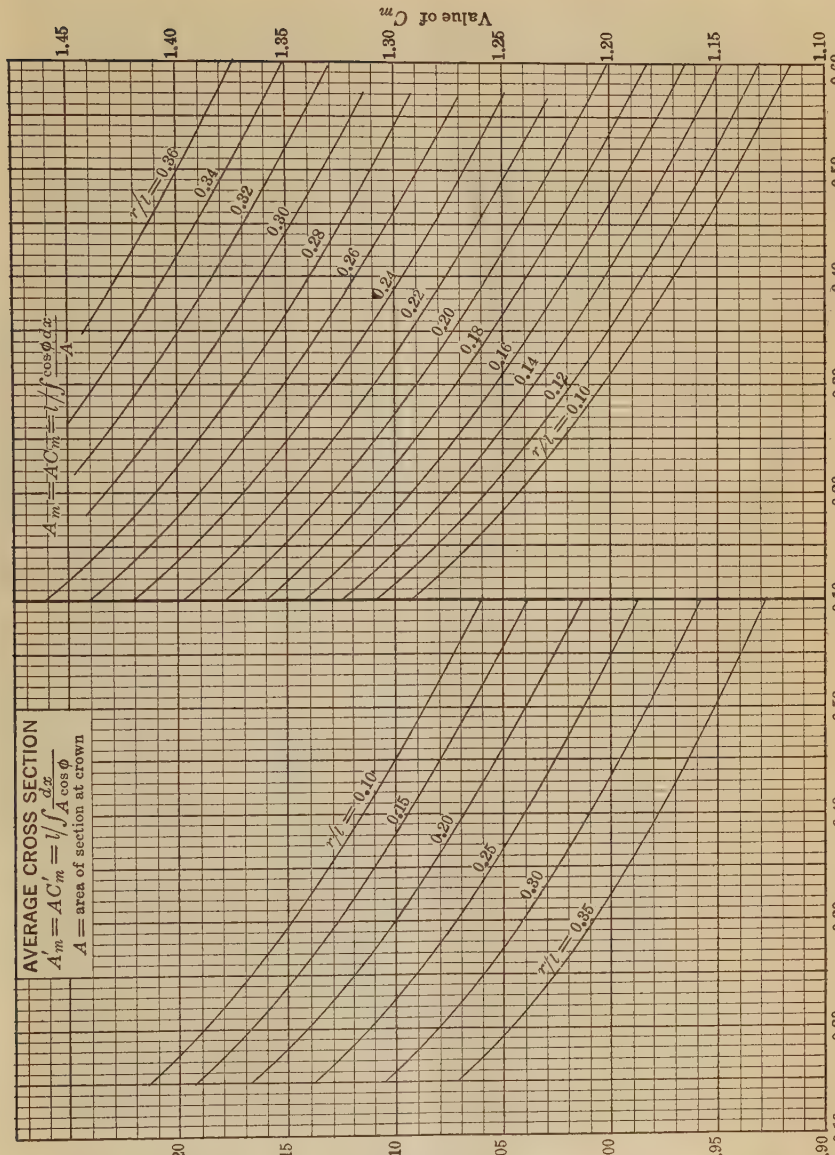
AVERAGE CROSS SECTION
$A'_m = AC'_m = l/\int \frac{dx}{A\cos\phi}$
A = area of section at crown
$A_m = AC_m = l/\int \frac{r\cos\phi\, dx}{A}$
$r/l = 0.36$
0.34
0.32
0.30
0.28
0.26
0.24
0.22
0.20
0.18
0.16
0.14
0.12
$r/l = 0.10$
$r/l = 0.10$
0.15
0.20
0.25
0.30
$r/l = 0.35$
Value of C_m
Value of C'_m
Value of m

FIG. 33.

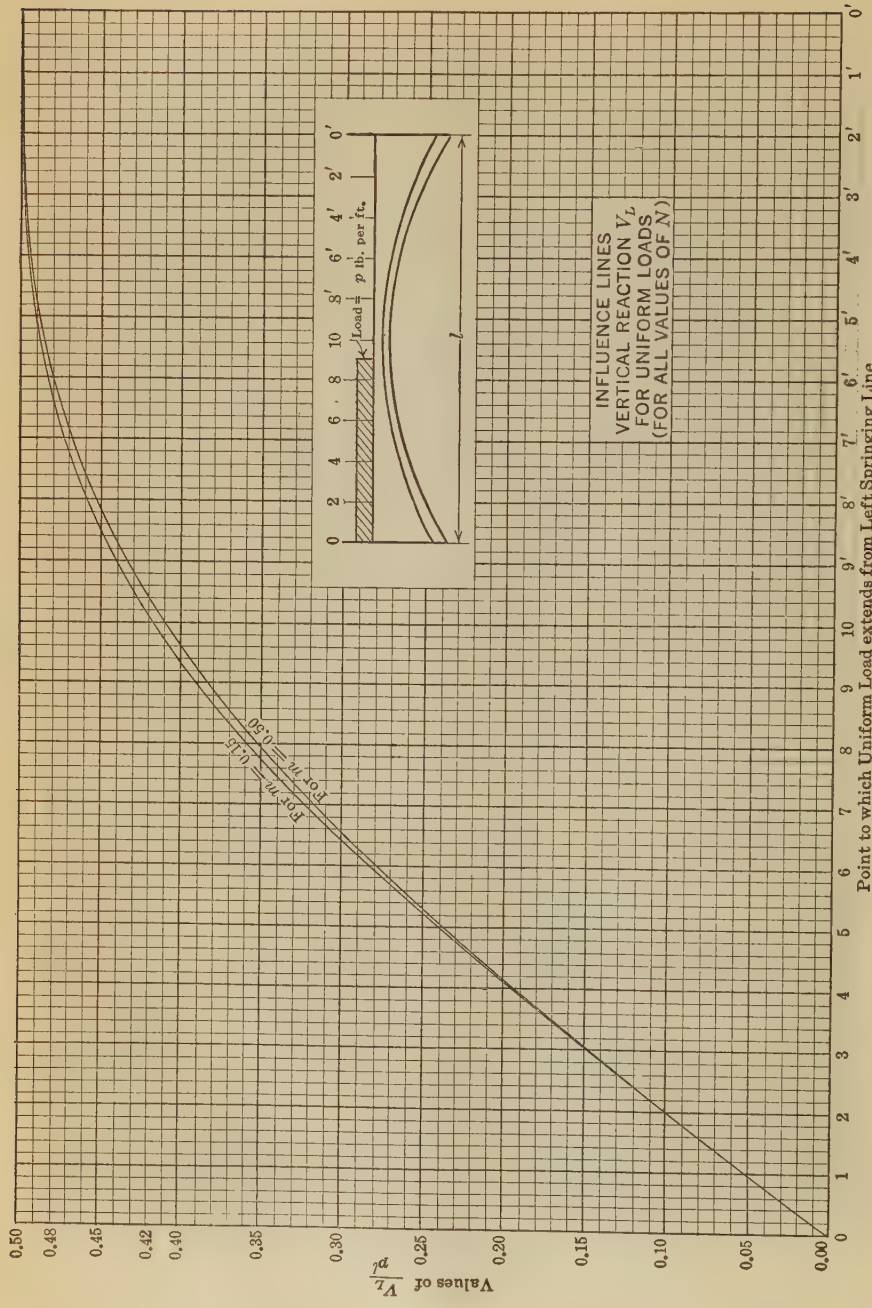
INFLUENCE LINES
VERTICAL REACTION V_L
FOR UNIFORM LOADS
(FOR ALL VALUES OF N)
Load = p lb. per ft.
l
For m = 0.35
For m = 0.50
Values of $\frac{V_L}{pl}$
Point to which Uniform Load extends from Left Springing Line

FIG. 34.

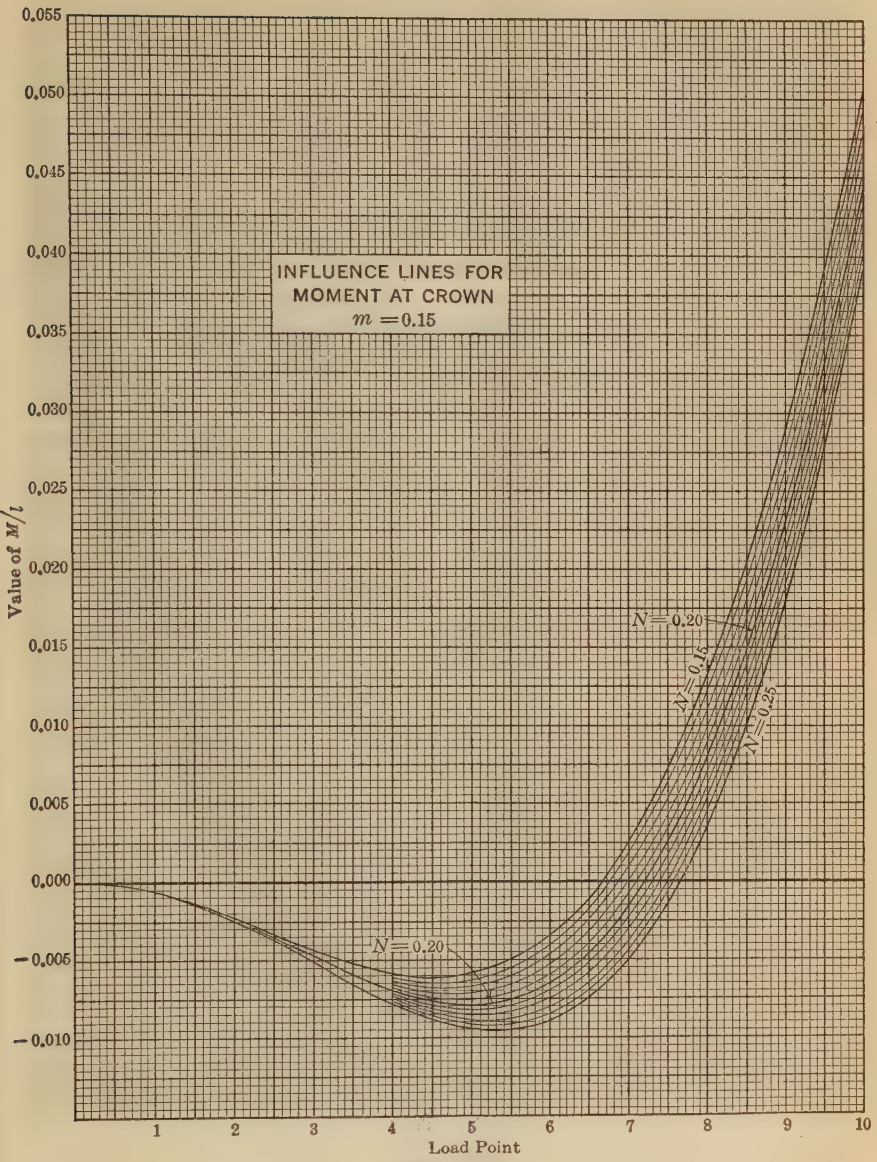
INFLUENCE LINES FOR
MOMENT AT CROWN
$m = 0.15$
Value of M/l
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
− 0.005
− 0.010
$N = 0.20$
$N = 0.15$
$N = 0.25$
$N = 0.20$
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 35.

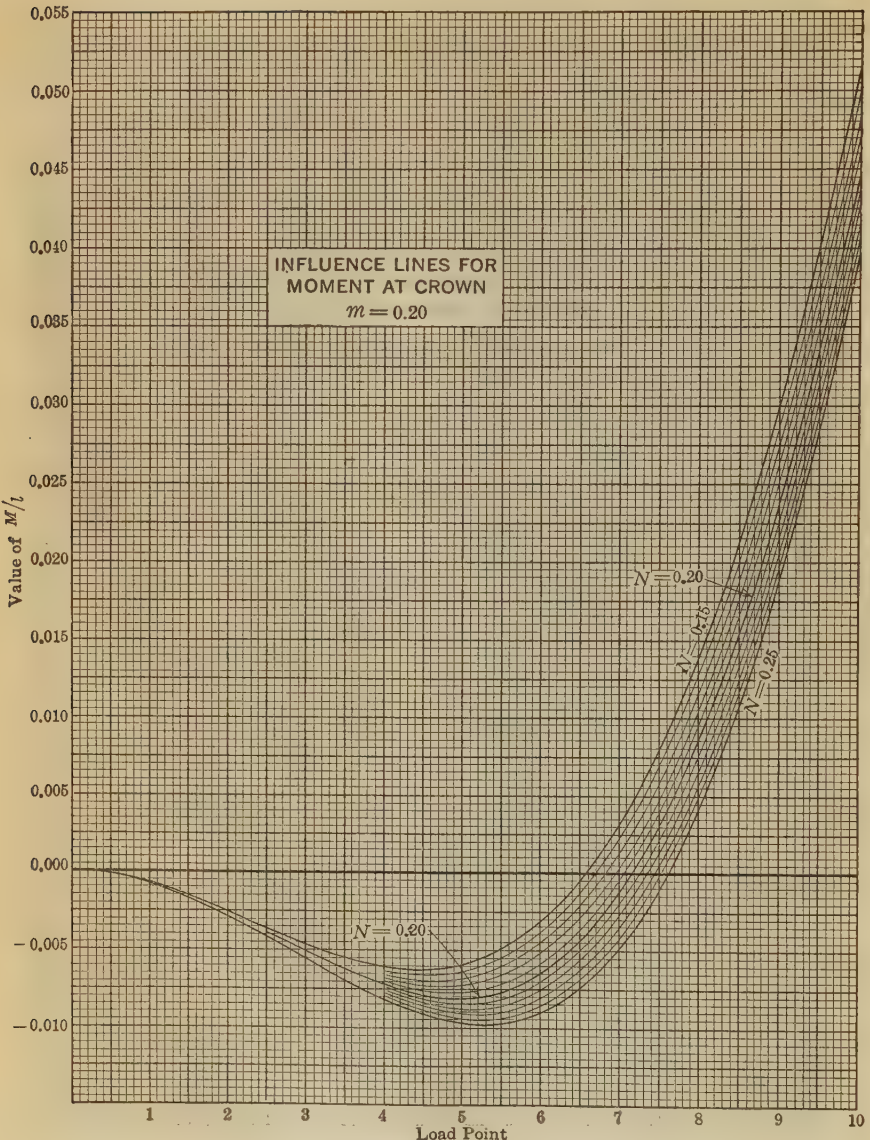
INFLUENCE LINES FOR
MOMENT AT CROWN
$m = 0.20$
Value of M/l
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
− 0.005
− 0.010
$N = 0.20$
$N = 0.15$
$N = 0.25$
$N = 0.20$
1
2
3
4
5
6
7
8
9
10
Load Point

Fig. 36.

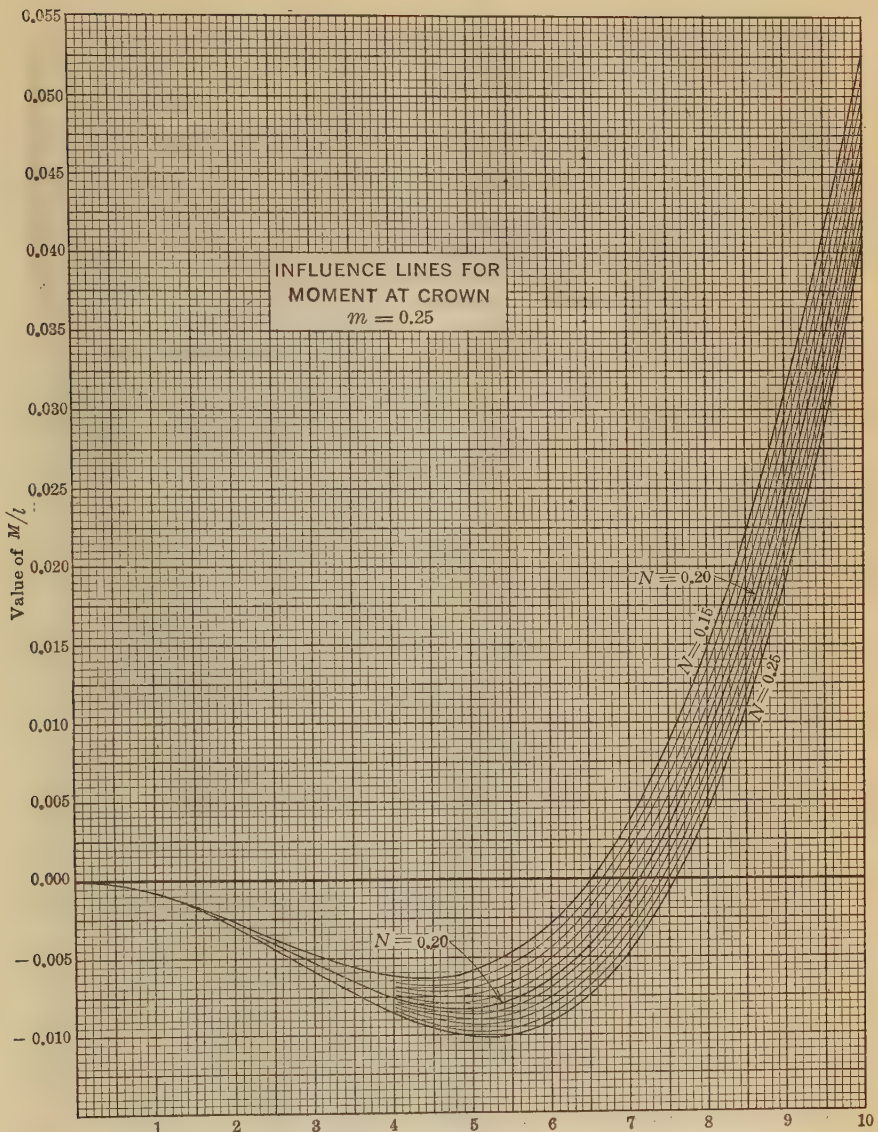
INFLUENCE LINES FOR
MOMENT AT CROWN
$m = 0.25$
Value of M/l
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
− 0.005
− 0.010
$N = 0.20$
$N = 0.15$
$N = 0.25$
$N = 0.20$
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 37.

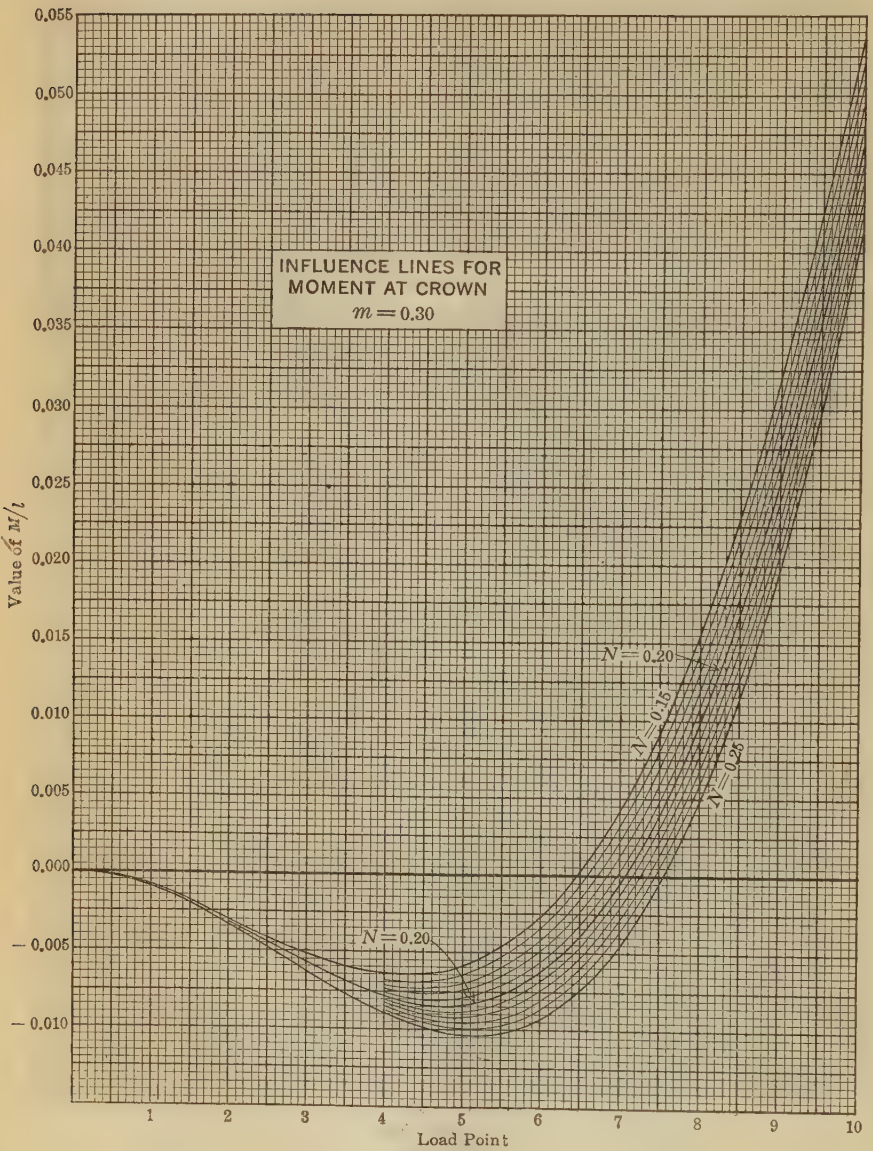
INFLUENCE LINES FOR
MOMENT AT CROWN
$m = 0.30$
Value of M/l
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
− 0.005
− 0.010
$N = 0.20$
$N = 0.15$
$N = 0.25$
$N = 0.20$
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 38.

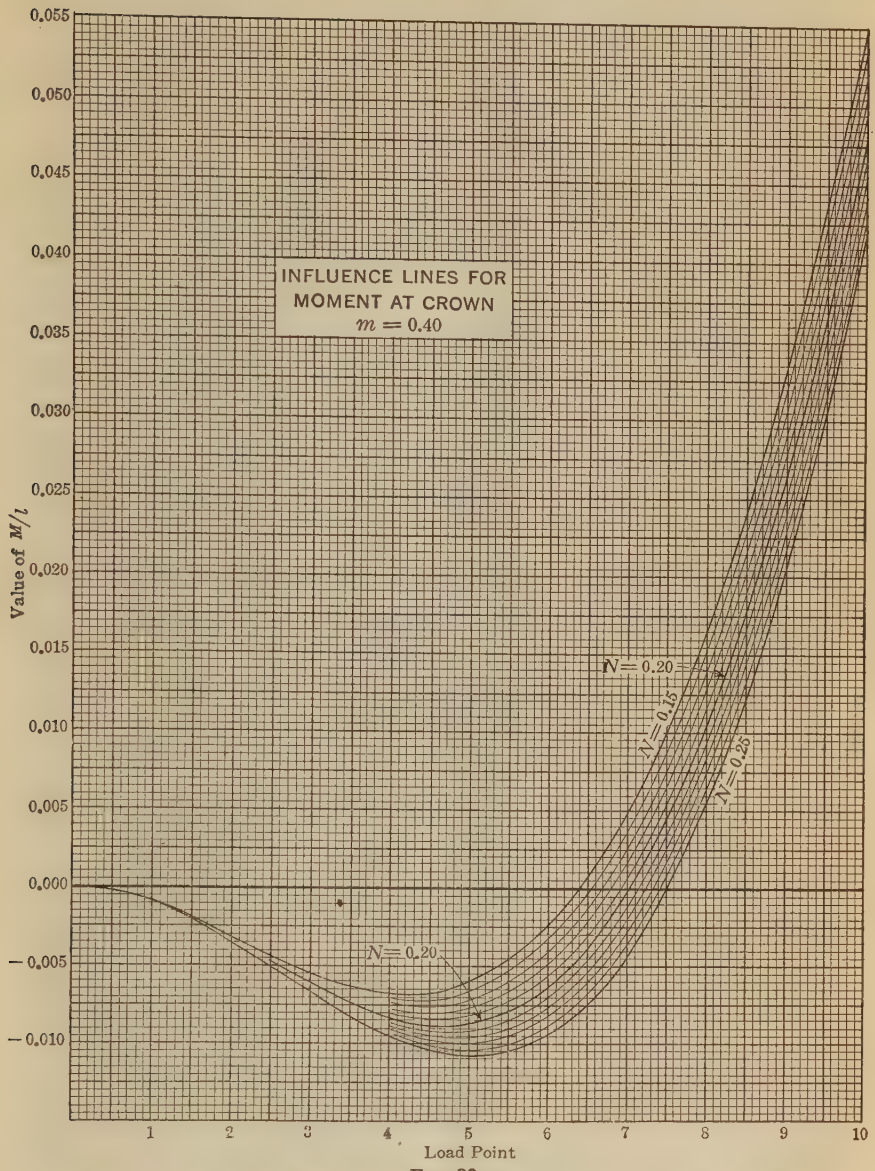
INFLUENCE LINES FOR
MOMENT AT CROWN
$m = 0.40$
Value of M/l
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
— 0.005
— 0.010
$N = 0.20$
$N = 0.15$
$N = 0.25$
$N = 0.20$
1
2
3
4
5
6
7
8
9
10
Load Point

FIG. 39.

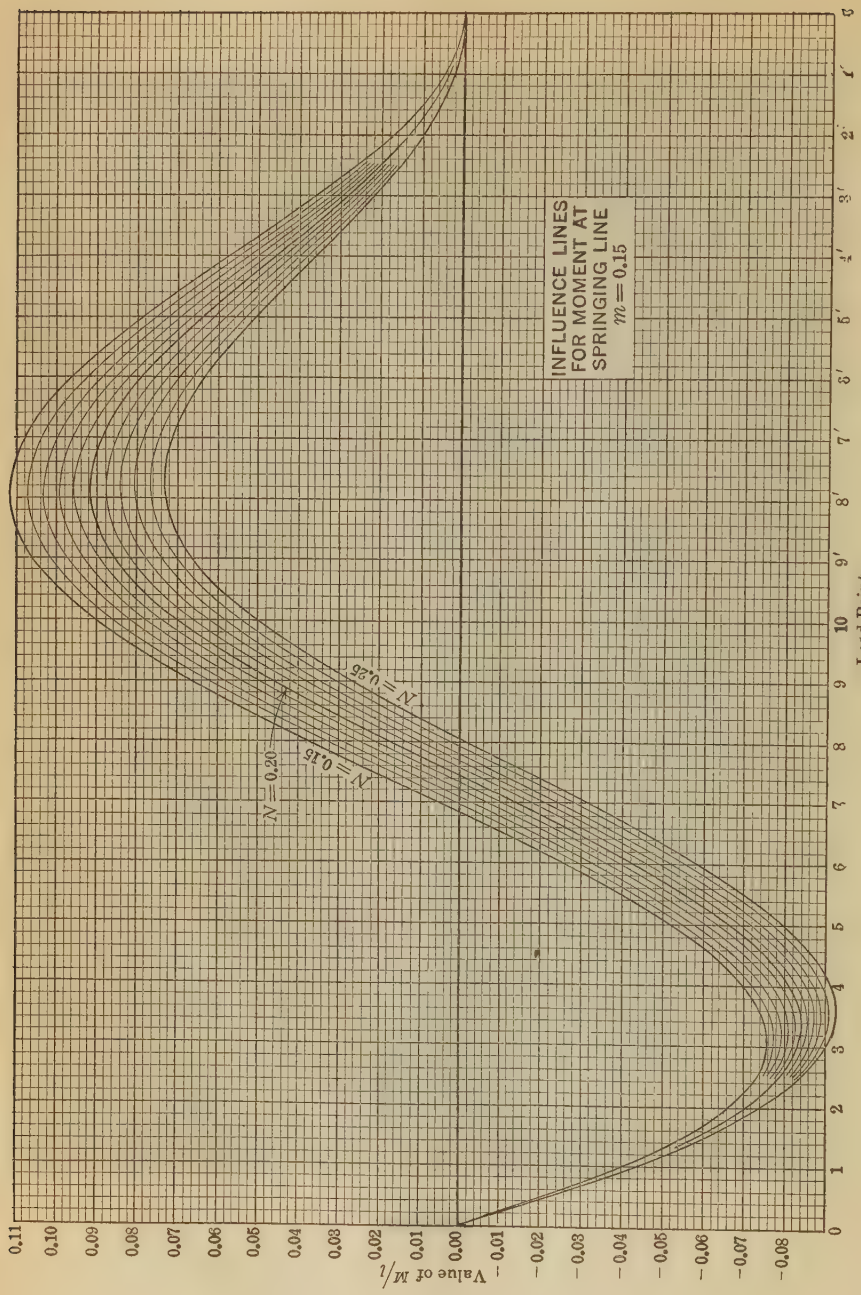
INFLUENCE LINES
FOR MOMENT AT
SPRINGING LINE
$m = 0.15$
$N = 0.25$
$N = 0.15$
$N = 0.20$
Value of M/l
0.11 0.10 0.09 0.08 0.07 0.06 0.05 0.04 0.03 0.02 0.01 0.00 − 0.01 − 0.02 − 0.03 − 0.04 − 0.05 − 0.06 − 0.07 − 0.08
0 1 2 3 4 5 6 7 8 9 10
0′ 1′ 2′ 3′ 4′ 5′ 6′ 7′ 8′ 9′
Load Point

FIG. 40.

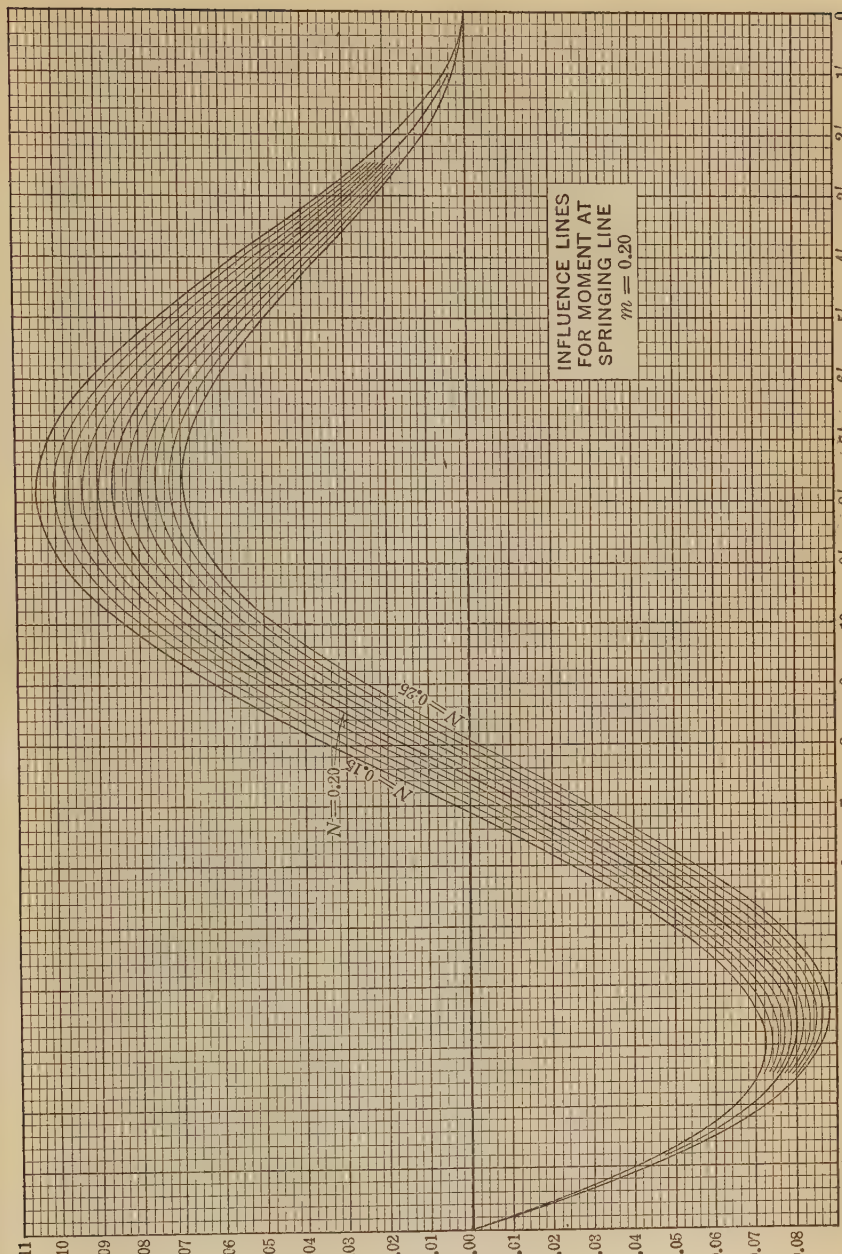
INFLUENCE LINES
FOR MOMENT AT
SPRINGING LINE
$m = 0.20$
$N = 0.25$
$N = 0.225$
$N = 0.20$
Value of M/l
0.11
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
− 0.01
− 0.02
− 0.03
− 0.04
− 0.05
− 0.06
− 0.07
− 0.08
0
1
2
3
4
5
6
7
8
9
10
9'
8'
7'
6'
5'
4'
3'
2'
1'
0
Load Point

FIG. 41.

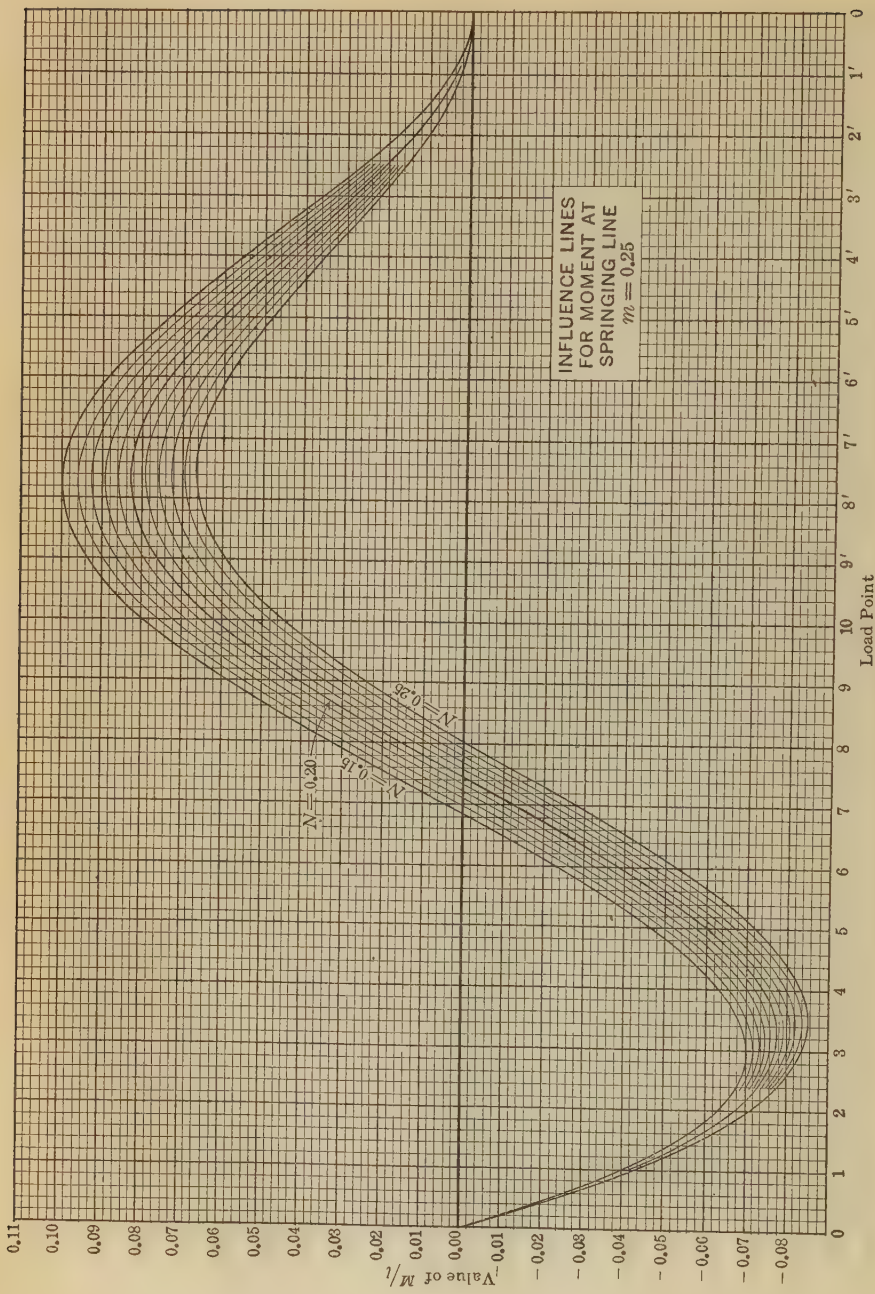
INFLUENCE LINES
FOR MOMENT AT
SPRINGING LINE
$m = 0.25$
$N = 0.35$
$N = 0.25$
$N = 0.20$
0.11 0.10 0.09 0.08 0.07 0.06 0.05 0.04 0.03 0.02 0.01 0.00 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08
Value of M/l
0 1 2 3 4 5 6 7 8 9 10
0 1' 2' 3' 4' 5' 6' 7' 8' 9'
Load Point

FIG. 42.

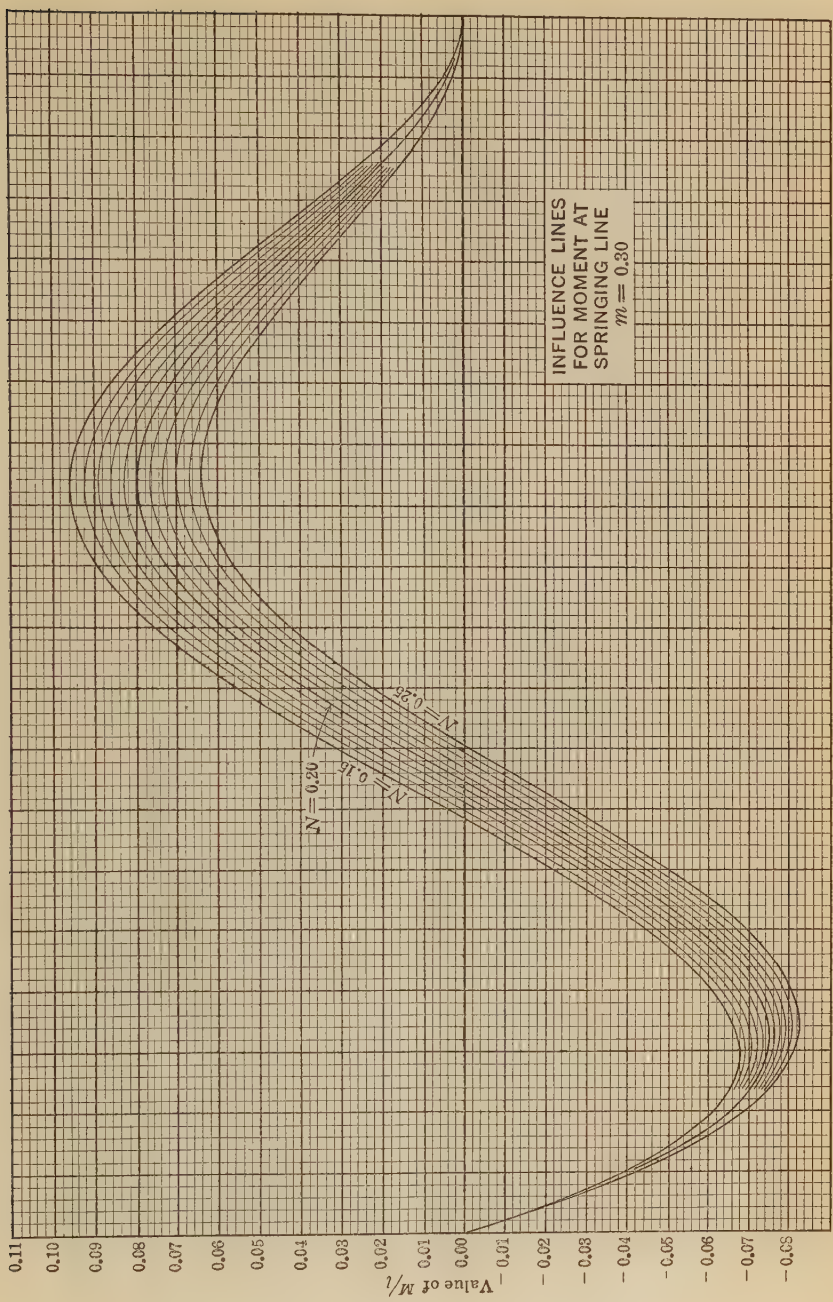
INFLUENCE LINES
FOR MOMENT AT
SPRINGING LINE
$m = 0.30$
$N = 0.25$
$N = 0.20$
$N = 0.15$
Value of M/l
Load Point
0.11 0.10 0.09 0.08 0.07 0.06 0.05 0.04 0.03 0.02 0.01 0.00 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08
0 1 2 3 4 5 6 7 8 9 10
0 1' 2' 3' 4' 5' 6' 7' 8' 9'

FIG. 43.

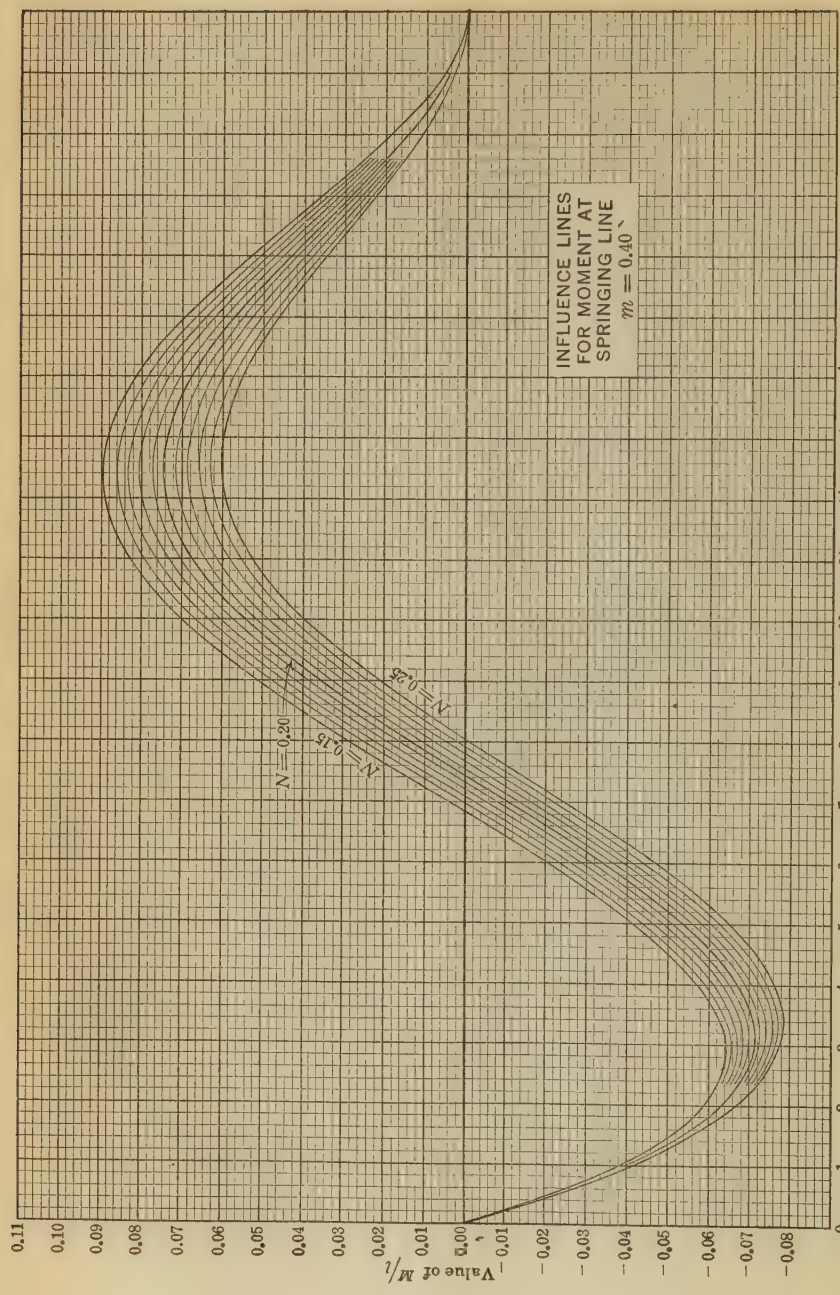
INFLUENCE LINES
FOR MOMENT AT
SPRINGING LINE
$m = 0.40$
$N = 0.25$
$N = 0.15$
$N = 0.20$
$N = 0$
Value of M/l
0.11 0.10 0.09 0.08 0.07 0.06 0.05 0.04 0.03 0.02 0.01 0.00 0.01 0.02 0.03 0.04 0.05 0.06 0.07 0.08
0 1 2 3 4 5 6 7 8 9 10
0 1' 2' 3' 4' 5' 6' 7' 8' 9'
Load Point

FIG. 44.

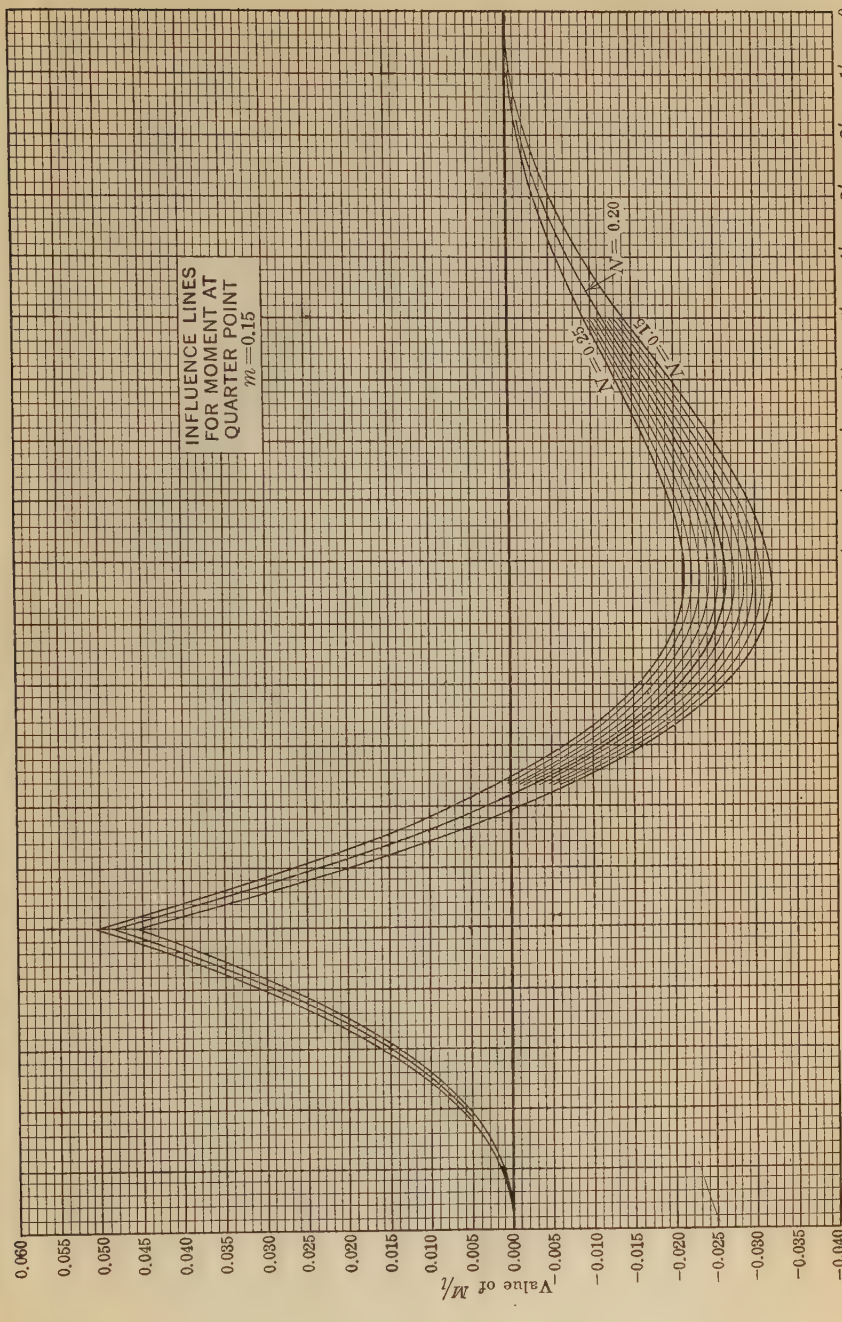
INFLUENCE LINES
FOR MOMENT AT
QUARTER POINT
$m = 0.15$
$N' = 0$
$N' = 0.05$
$N' = 0.10$
$N' = 0.15$
$N' = 0.20$
Value of M/l
0.060
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
0.005
−0.010
−0.015
−0.020
−0.025
−0.030
−0.035
−0.040
0
1
2
3
4
5
6
7
8
9
10
Load Points
0
1′
2′
3′
4′
5′
6′
7′
8′
9′

FIG. 45.

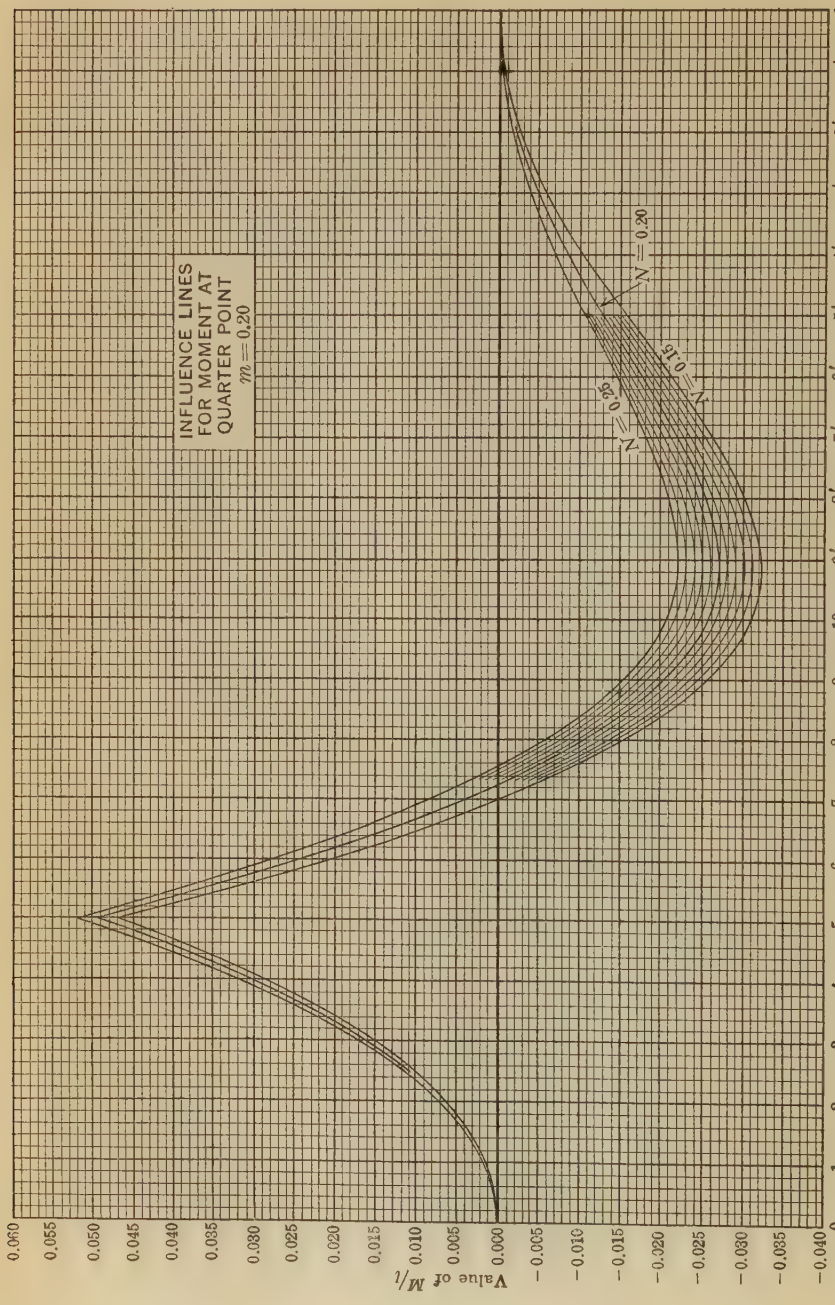
INFLUENCE LINES
FOR MOMENT AT
QUARTER POINT
$m = 0.20$
$N = 0.20$
$N = 0.18$
$N = 0.16$
Value of M/l
0.060
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
− 0.005
− 0.010
− 0.015
− 0.020
− 0.025
− 0.030
− 0.035
− 0.040
0 1 2 3 4 5 6 7 8 9 10
0 1′ 2′ 3′ 4′ 5′ 6′ 7′ 8′ 9′
Load Points

FIG. 46.

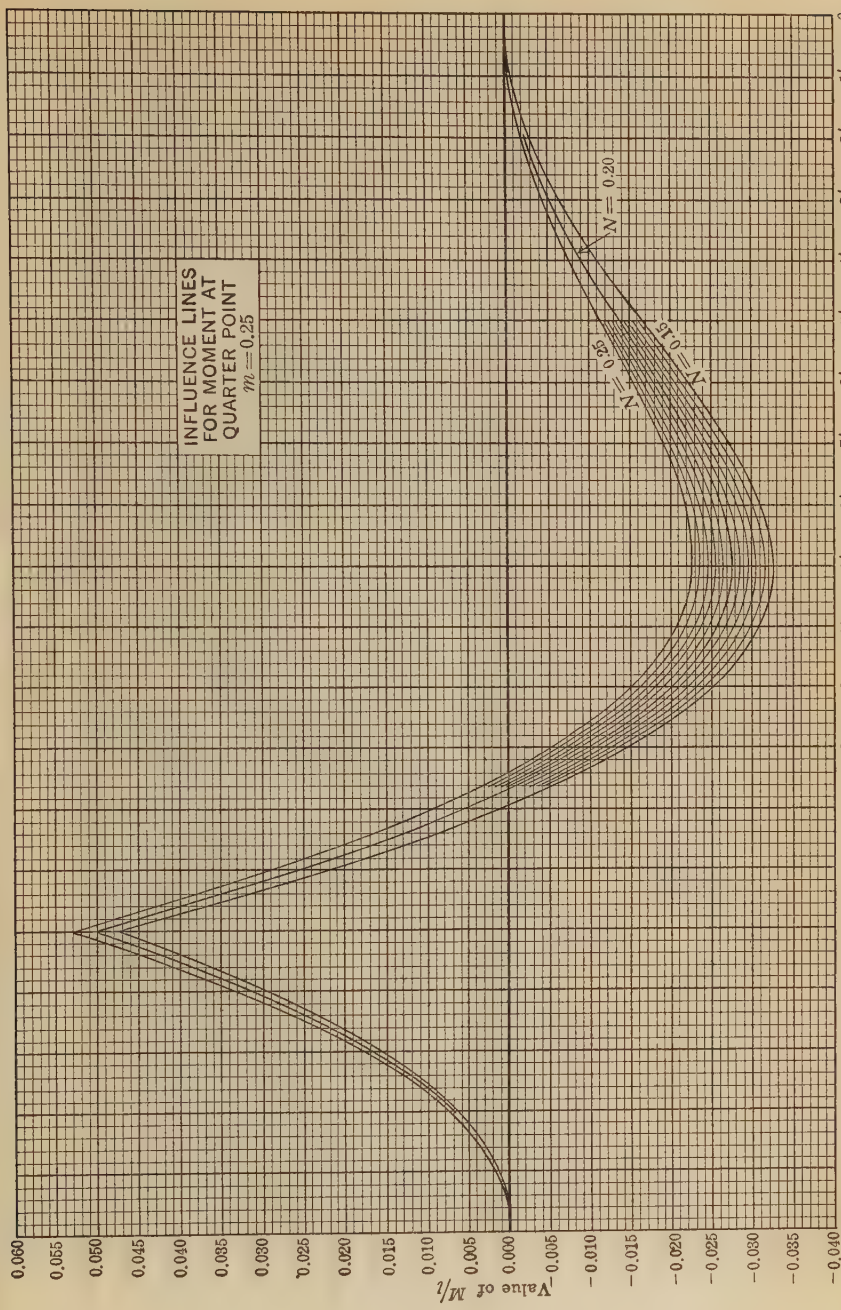
INFLUENCE LINES
FOR MOMENT AT
QUARTER POINT
$m = 0.25$
$N = 0.20$
$N = 0.15$
$N = 0.25$
Value of M/l
0.060
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
−0.005
−0.010
−0.015
−0.020
−0.025
−0.030
−0.035
−0.040
0 1 2 3 4 5 6 7 8 9 10
0 1′ 2′ 3′ 4′ 5′ 6′ 7′ 8′ 9′
Load Points

FIG. 47.

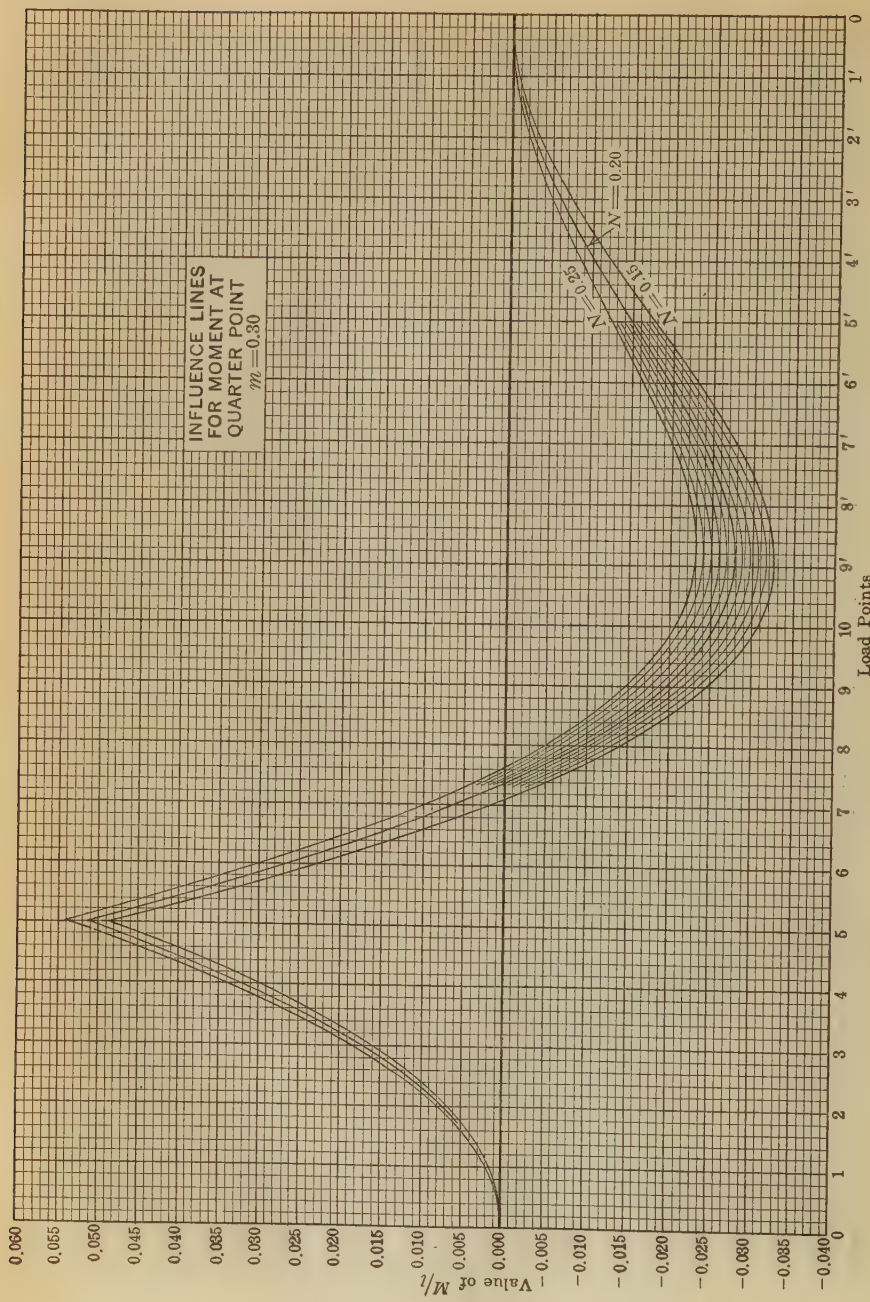
INFLUENCE LINES
FOR MOMENT AT
QUARTER POINT
$m=0.30$
$N=0.35$
$N=0.15$
$N=0.20$
Value of M/l
0.060
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
0.005
0.010
0.015
0.020
0.025
0.030
0.035
0.040
0 1 2 3 4 5 6 7 8 9 10
0 1' 2' 3' 4' 5' 6' 7' 8' 9'
Load Points

FIG. 48.

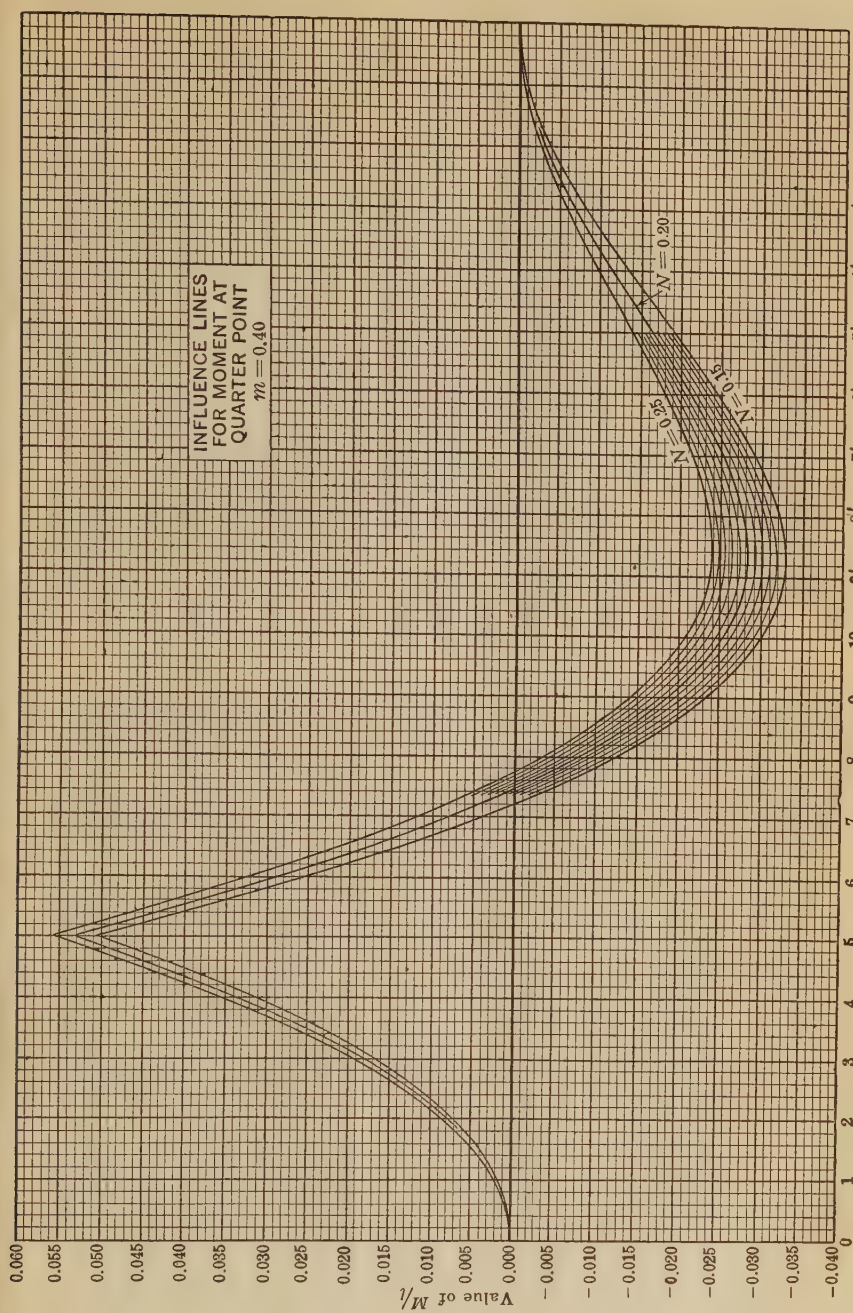
INFLUENCE LINES
FOR MOMENT AT
QUARTER POINT
$m = 0.40$
$N = 0.20$
$N = 0.15$
$N = 0.10$
$N = 0.05$
$N = 0.00$
Value of M/l
0.060
0.055
0.050
0.045
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0.000
−0.005
−0.010
−0.015
−0.020
−0.025
−0.030
−0.035
−0.040
0 1 2 3 4 5 6 7 8 9 10
0 1′ 2′ 3′ 4′ 5′ 6′ 7′ 8′ 9′
Load Points

FIG. 49.

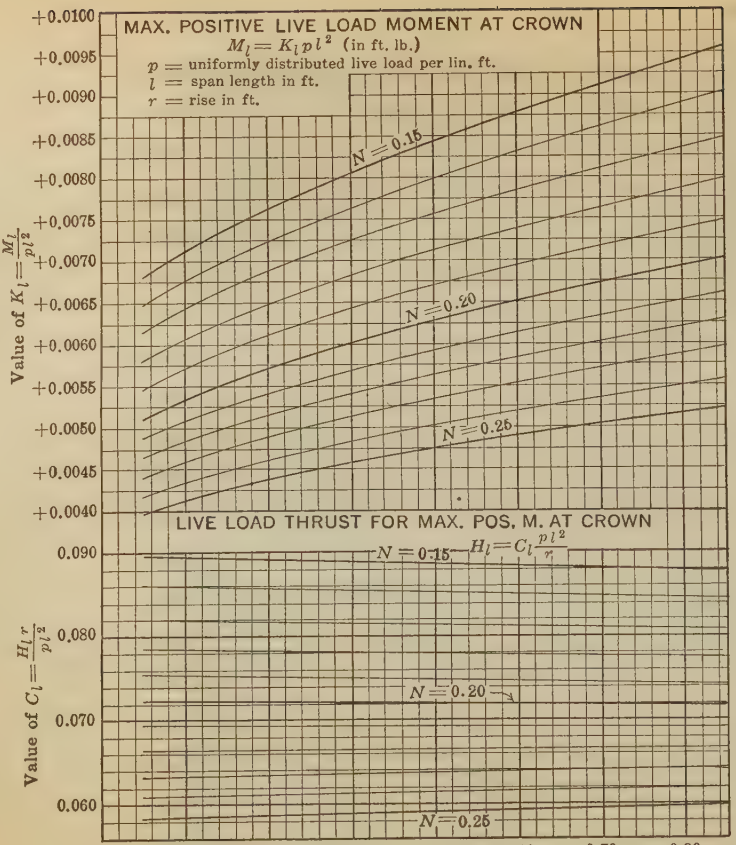
MAX. POSITIVE LIVE LOAD MOMENT AT CROWN
$M_l = K_l p l^2$ (in ft. lb.)
p = uniformly distributed live load per lin. ft.
l = span length in ft.
r = rise in ft.
N = 0.15
N = 0.20
N = 0.25
Value of $K_l = \frac{M_l}{pl^2}$
+0.0100
+0.0095
+0.0090
+0.0085
+0.0080
+0.0075
+0.0070
+0.0065
+0.0060
+0.0055
+0.0050
+0.0045
+0.0040
LIVE LOAD THRUST FOR MAX. POS. M. AT CROWN
$H_l = C_l \frac{pl^2}{r}$
N = 0.15
N = 0.20
N = 0.25
Value of $C_l = \frac{H_l r}{pl^2}$
0.090
0.080
0.070
0.060
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
Value of m

FIG. 50.

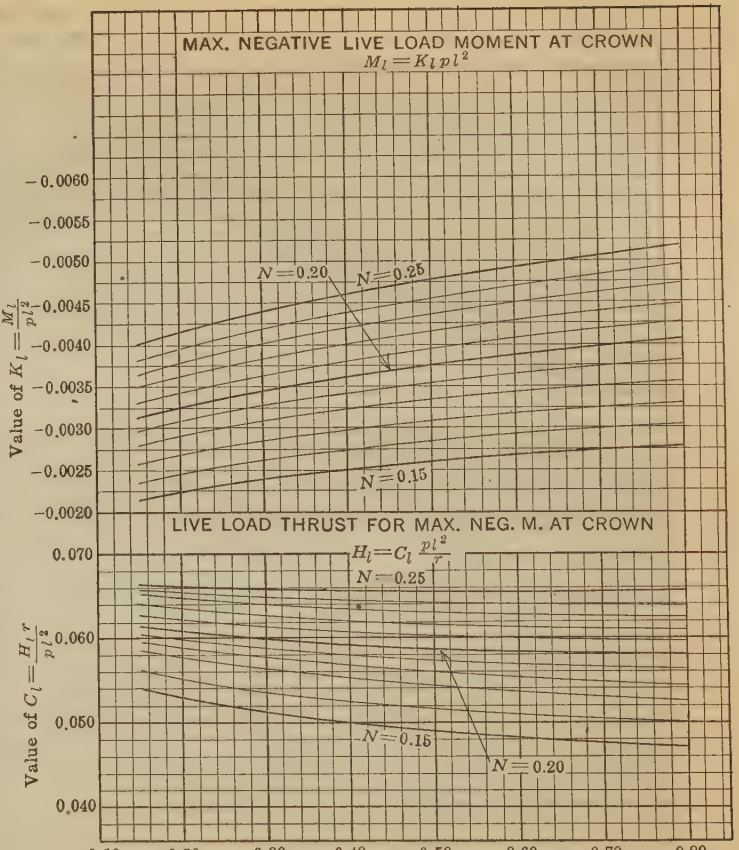
MAX. NEGATIVE LIVE LOAD MOMENT AT CROWN
$M_l = K_l p l^2$
N = 0.20
N = 0.25
N = 0.15
Value of $K_l = \frac{M_l}{pl^2}$
−0.0060
−0.0055
−0.0050
−0.0045
−0.0040
−0.0035
−0.0030
−0.0025
−0.0020
LIVE LOAD THRUST FOR MAX. NEG. M. AT CROWN
$H_l = C_l \frac{pl^2}{r}$
N = 0.25
N = 0.15
N = 0.20
Value of $C_l = \frac{H_l r}{pl^2}$
0.070
0.060
0.050
0.040
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
Value of m

FIG. 51.

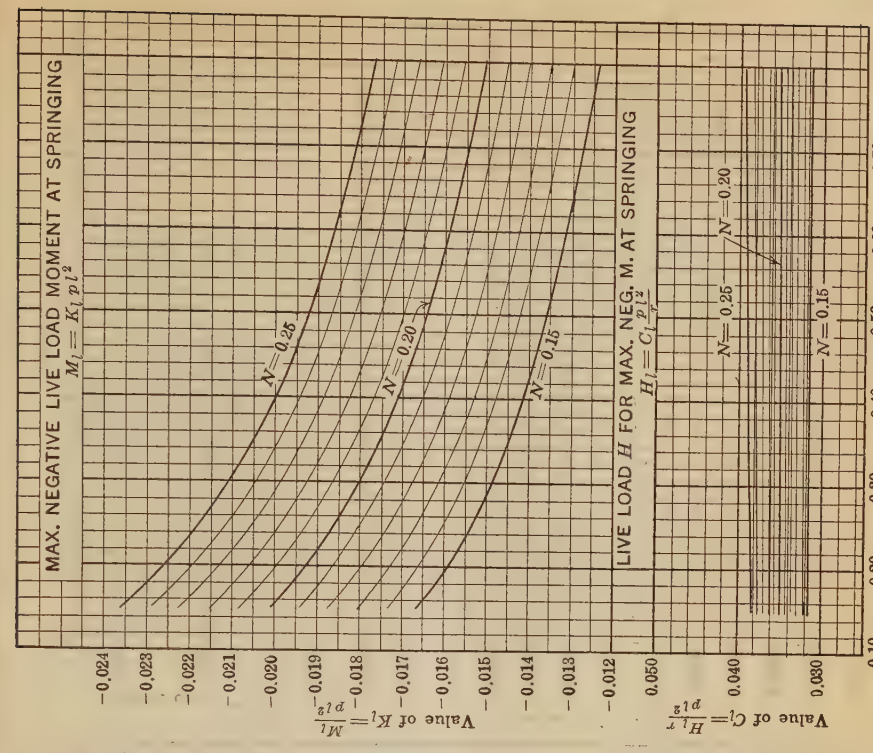
MAX. NEGATIVE LIVE LOAD MOMENT AT SPRINGING
$M_l = K_l\, p l^2$
LIVE LOAD H FOR MAX. NEG. M. AT SPRINGING
$H_l = C_l \frac{p l^2}{r}$
$N = 0.25$
$N = 0.20$
$N = 0.15$
Value of $K_l = \frac{M_l}{p l^2}$
Value of $C_l = \frac{H_l\, r}{p l^2}$
Value of m
−0.024
−0.023
−0.022
−0.021
−0.020
−0.019
−0.018
−0.017
−0.016
−0.015
−0.014
−0.013
−0.012
0.050
0.040
0.030
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80

FIG. 53.

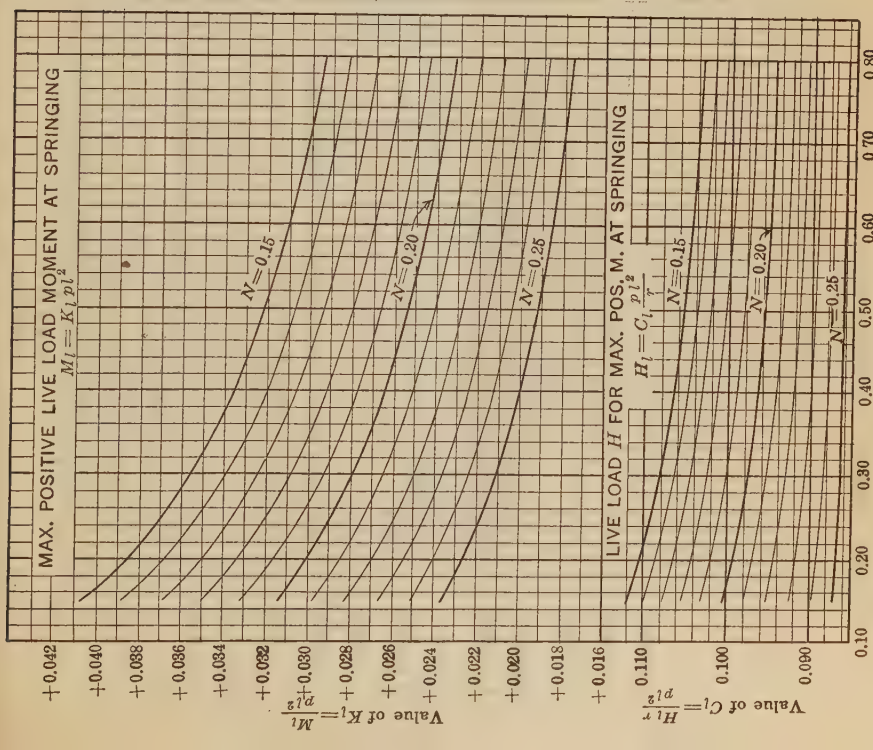
MAX. POSITIVE LIVE LOAD MOMENT AT SPRINGING
$M_l = K_l\, p l^2$
LIVE LOAD H FOR MAX. POS. M. AT SPRINGING
$H_l = C_l \frac{p l^2}{r}$
$N = 0.15$
$N = 0.20$
$N = 0.25$
Value of $K_l = \frac{M_l}{p l^2}$
Value of $C_l = \frac{H_l\, r}{p l^2}$
Value of m
+0.042
+0.040
+0.038
+0.036
+0.034
+0.032
+0.030
+0.028
+0.026
+0.024
+0.022
+0.020
+0.018
+0.016
0.110
0.100
0.090
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80

FIG. 52.

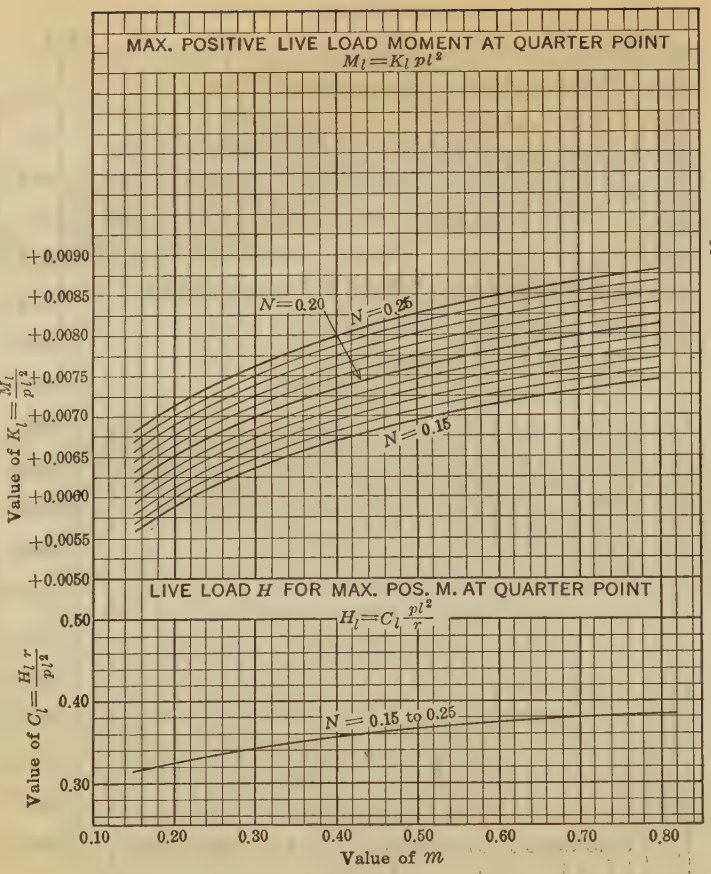
MAX. POSITIVE LIVE LOAD MOMENT AT QUARTER POINT
$M_l = K_l\, pl^2$
Value of $K_l = \frac{M_l}{pl^2}$
+0.0090
+0.0085
+0.0080
+0.0075
+0.0070
+0.0065
+0.0060
+0.0055
+0.0050
N = 0.20
N = 0.25
N = 0.15
LIVE LOAD H FOR MAX. POS. M. AT QUARTER POINT
$H_l = C_l \frac{pl^2}{r}$
Value of $C_l = \frac{H_l\, r}{pl^2}$
0.50
0.40
0.30
N = 0.15 to 0.25
0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80
Value of m

FIG. 54.

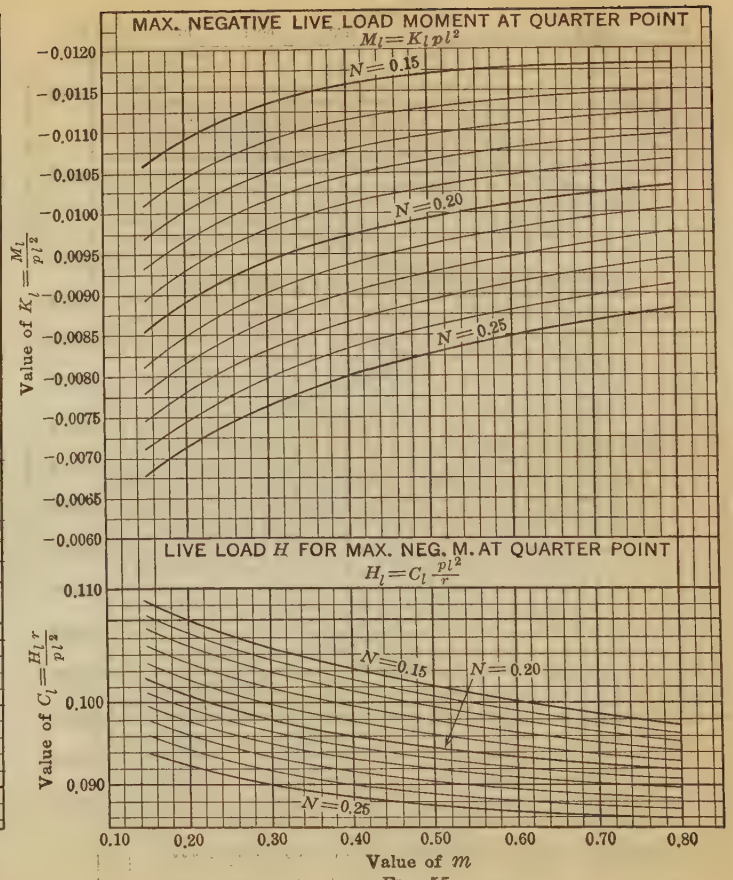
MAX. NEGATIVE LIVE LOAD MOMENT AT QUARTER POINT
$M_l = K_l\, pl^2$
Value of $K_l = \frac{M_l}{pl^2}$
−0.0120
−0.0115
−0.0110
−0.0105
−0.0100
−0.0095
−0.0090
−0.0085
−0.0080
−0.0075
−0.0070
−0.0065
−0.0060
N = 0.15
N = 0.20
N = 0.25
LIVE LOAD H FOR MAX. NEG. M. AT QUARTER POINT
$H_l = C_l \frac{pl^2}{r}$
Value of $C_l = \frac{H_l\, r}{pl^2}$
0.110
0.100
0.090
N = 0.15
N = 0.20
N = 0.25
0.10 0.20 0.30 0.40 0.50 0.60 0.70 0.80
Value of m

FIG. 55.

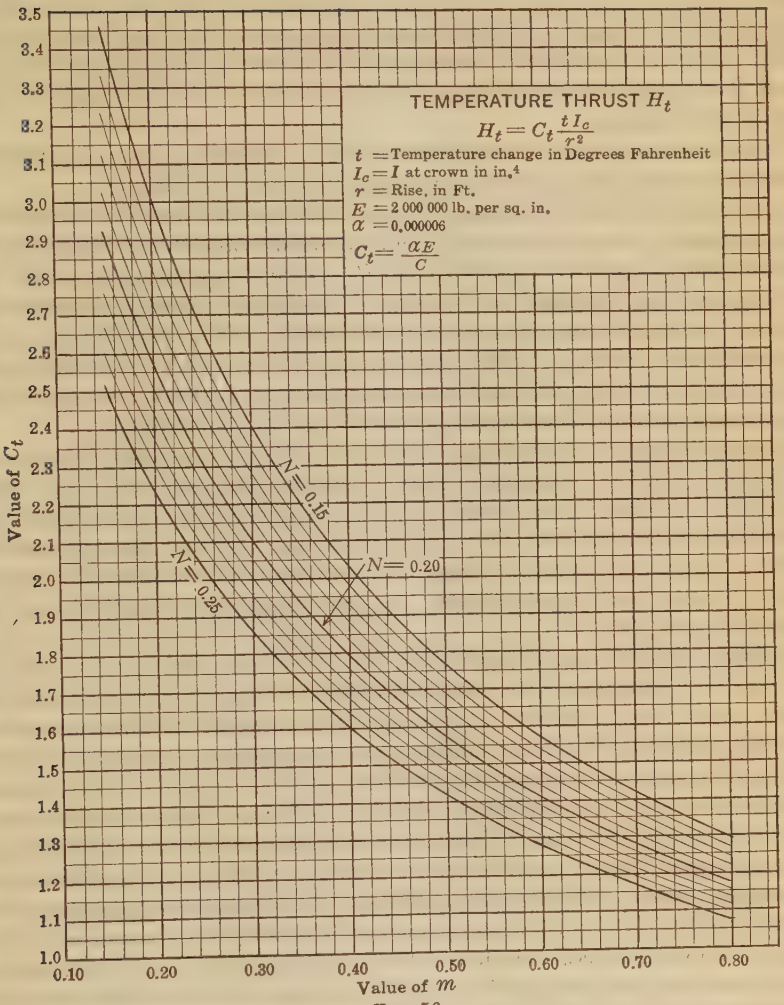
TEMPERATURE THRUST H_t
$H_t = C_t \frac{t I_c}{r^2}$
t = Temperature change in Degrees Fahrenheit
$I_c = I$ at crown in in.^{4}
r = Rise, in Ft.
E = 2 000 000 lb. per sq. in.
α = 0.000006
$C_t = \frac{\alpha E}{C}$
N = 0.15
N = 0.20
N = 0.25
Value of C_t
Value of m
3.5
3.4
3.3
3.2
3.1
3.0
2.9
2.8
2.7
2.6
2.5
2.4
2.3
2.2
2.1
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80

FIG. 56.

DISCUSSION

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The analysis of symmetrical concrete arches, as presented by the author, is the most complete that has appeared. After deducing formulas that apply to any arch rib, a formula for the arch axis is derived, based on a certain assumption as to the increase of dead load from the crown to the springing line, which, the author claims, coincides "almost exactly with the dead load pressure line for practical bridge arches". As the subsequent development is based on this assumption, it is hoped that designers with sufficient data will give it a thorough test and publish their results.

The assumption leads to simple formulas for computing the co-ordinates of points on the arch axis and for ascertaining the dead load reactions.

A formula has been assumed, likewise, for the thickness of the arch; it gives a pleasing contour and leads by an extended analysis to formulas for the reactions, M', V_L, and H, for a live load, $P = 1$, at any point of the arch. The formulas have been checked by the writer. Their use is greatly facilitated by the many tables and diagrams given in the paper. Also, the diagrams giving maximum and minimum bending moments, with corresponding thrusts, for the crown, springing line, and quarter-points, together with temperature and rib-shortening effects, will be especially appreciated by the engineer.

Taking the origin at the elastic center simplifies the general formulas for any arch. Its main advantage is that it enables the solution to be readily effected by the use of equilibrium polygons, whether the arch axis has been divided into parts such that Δx is constant or such that Δw remains the same for each supposed voussoir. When Δx is taken as constant, the formulas involve a great deal of computation, so that the graphical method has been exclusively used hitherto.

However, when the axis is divided so that Δw is constant for the successive voussoirs the formulas can be reduced to simple forms, readily computed and giving precise results which, afterward, can be checked by the approximate graphical methods. Graphical solutions are approximate, for each line drawn involves a possible error in length, direction, or intersection, so that the result rarely checks with the true position when ten or twenty lines have to be drawn.

The simplified formulas will be deduced subsequently and applied to the particular arch, Fig. 19, not only to illustrate the method of computation, but to compare results with those found by the author, where the division of the neutral axis was such that Δx was constant. The points on the arch axis given in Fig. 19 were plotted, the curve of the axis sketched in by eye, and its length measured and found to be 53.42 ft. A scale of 2 ft. = 1 in. was used. To this scale, lay off, in Fig. 57, $A\,C = 53.42$ ft. and draw the I-curve by the aid of Table 1, taking for convenience, 0.001 of the tabular value. To divide the length, $A\,C$, from the springing line, A, to the crown, C, into ten parts, Δs_1, Δs_2, ..., Δs_{10}, such that:

$$\frac{\Delta s_1}{I_1} = \frac{\Delta s_2}{I_2} = , \ldots = \frac{\Delta s_{10}}{I_{10}} = \text{constant}$$

* Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

I_1 being 0.001 of the moment of inertia at the mid-point of Δs_1, etc., first assume Δs_1, measure I_1 at its mid-point, and compute $\frac{\Delta s_1}{I_1}$. Then, at the end of Δs_1, draw Line 2 parallel to Line 1 to an intersection with the I-curve, measure I_2, and compute $\Delta s_2 = \left(\frac{\Delta s_1}{I_1}\right) I_2$ and lay it off. By continuing thus to compute, Δs_2, Δs_3, ..., the dotted parallel lines need never be drawn, so that nearly half the error of the usual construction is avoided, which is an important practical consideration.

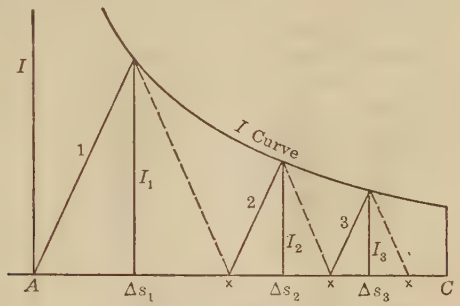

FIG. 57.

For the arch in question, on the second trial, Δs_1 was assumed as equal to 10.72 ft., and the construction gave for ten divisions, $\Delta s_1 + \Delta s_2 + \ldots + \Delta s_{10} = 53.80$ ft., or 0.38 ft. in excess of $A\ C$. Assuming a proportionate decrease in Δs_1, it must be shortened by $\frac{0.38}{53.8}$ (10.72) = 0.08 ft. Thus, for the third trial, Δs_1 will be assumed to have the length, 10.72 — 0.08 = 10.64 ft. When the excess is much larger, it is well to take Δs_1 smaller than this proportionate rule gives. To proceed: With Δs_1 equal to 10.64 ft., at $\frac{1}{2}\Delta s_1$, measure $I_1 = 20.4 \ldots \frac{\Delta s_1}{I_1} = 0.521$. Hence,

$$\frac{\Delta s_2}{I_2} = \frac{\Delta s_3}{I_3} = \ldots, = 0.521$$

Thus, as before, at the end of Δs_1, draw Line 2 parallel to Line 1, measure I_2, and compute $\Delta s_2 = 0.521\, I_2$. Continue thus to compute and lay off, Δs_3, Δs_4, ..., Δs_{10}, only drawing the full lines, and find $\Delta s_1 + \ldots + \Delta s_{10} = 53.40$ ft. The difference, 53.42 — 53.40 = 0.02 ft., can now be distributed between any two divisions. It is hoped that the writer may be pardoned for entering into this detail, as the subject seems to offer difficulties to some who spend an unnecessary amount of time on this division of the arch axis.

The lengths, Δs_1, $\Delta s_1 + \Delta s_2$, ..., thus ascertained, are laid off on the arch axis, beginning at the springing line, thus dividing it into parts such that $\frac{\Delta s}{I}$ is the same for each voussoir. The centers of these parts are then marked, 1, 2, ... 10, 10′, ... 2′, 1′, as shown in Fig. 20. The co-ordinates, x, y_0, referred to the crown as origin, are then scaled off. Since Δw is constant, the

sum of the y_0's for the half arch divided by 10 gives the distance below C, the center of the crown section, to "the elastic center" as 3.093 ft. The author's value is 3.14 ft., and it will be found that this is the main discrepancy in the two methods, since it affects the value of all the y's.

Taking the origin now at the elastic center, the x-axis horizontal and the y-axis vertical, $y = 3.093 - y_0$. The values of x and y for the points $1', 2' \ldots, 10'$, are inserted in Table 21, together with their squares and products.

TABLE 21.

Point. (1)	y. (2)	y^2. (3)	x. (4)	x^2. (5)	$x\,y$. (6)
1′	−8.71	75.86	45.80	2 098	−398.92
2′	−4.23	17.89	37.94	1 439	−160.49
3′	−1.60	2.56	31.56	996	− 50.50
4′	+0.08	0.01	25.96	674	+ 2.08
5′	+1.21	1.46	21.00	441	+ 25.41
6′	+1.93	3.72	16.64	277	+ 32.12
7′	+2.46	6.05	12.58	158	+ 30.95
8′	+2.80	7.84	8.75	77	+ 24.50
9′	+3.00	9.00	5.15	27	+ 15.45
10′	+3.06	9.36	1.70	3	+ 5.20
		133.75	207.08	6 190	−609.91
		2		2	+135.71
.....	$\Sigma_B^C y = 0$.	267.50		$\Sigma_B^A x^2 = 12\,380$	$\Sigma_B^C x\,y = -474.20$

The formula for M' is (page 951):

$$M' = P \frac{\sum_B^P x' \,\Delta w}{2 \sum_B^C \Delta w}$$

in which, $\Delta w = \dfrac{\Delta s}{E I}$ is constant. (See Fig. 58.)

Let

N = total number of voussoirs in the arch;

n = number of points, such as $1', 2' \ldots$, between B and the load, $P = 1$;

d = horizontal distance from the crown to P;

x' = horizontal distance from P to points such as $1', 2', \ldots$, to its right.

$$\therefore 2 \sum_B^C \Delta w = N \,.\, \Delta w\,; \; x' = x - d$$

$$\therefore M' = \frac{1}{N} \sum_B^P x' = \frac{1}{N} \sum_B^P (x - d)$$

Designating the co-ordinates, x, y, of the points, $1', 2', \ldots, 10'$, as (x_1, y_1), (x_2, y_2), $\ldots$ (x_{10}, y_{10}):

$$\sum_B^P (x - d) = (x_1 - d) + (x_2 - d) + \ldots + (x_n - d)$$

$$\therefore M' = \frac{(x_1 + x_2 + \ldots + x_n) - n\,d}{N}$$

Thus, if $P = 1$ is placed successively at the points, 1′, 2′, ..., 10′, d takes the successive values, x_1, x_2, ..., x_{10}, and n, the successive values, 0, 1, 2, ..., 9. (See Fig. 58.)

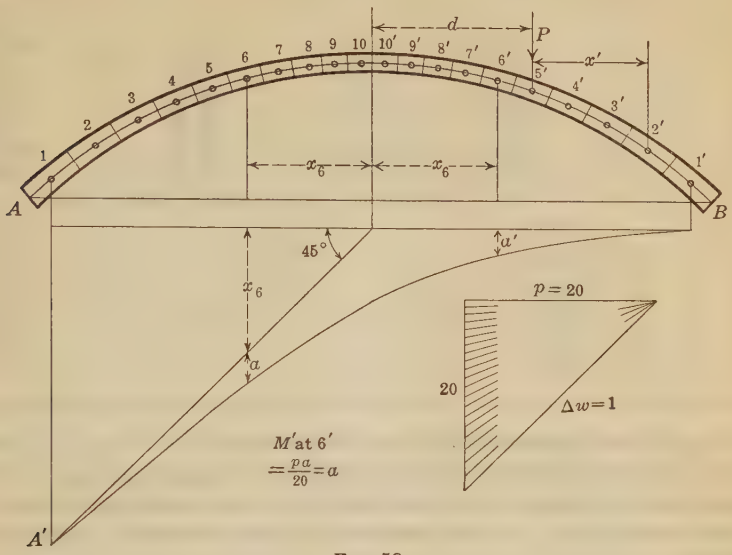

FIG. 58.

No mistake can be made in writing the value of n if Fig. 58 is noted. It does not include the point where P is placed. In reality, P is a distributed load, so that in case the equilibrium polygons of Fig. 17 are drawn, they should be rounded off to form a smooth curve to correspond to infinitesimal divisions. The same remark applies to the graph of any of the derived quantities.

If the equilibrium polygon, 4′, of Fig. 17 is drawn with the pole distance, p_4, equal to the sum of the loads, $\Sigma\,\Delta\,w$, the end tangents make an angle of 45° with each other, as in Fig. 58, and a vertical ordinate from the tangent at the right to the curve, to the scale of distance, is equal to M'. Thus, for a load, 1, at the point, 6′, $M' = a'$. For a load, 1, at the symmetrical point, 6, $M' = x_6 + a = x_6 + a'$, since $a = a'$. By this principle, from the values of M' already computed for the points, 1′, ..., 10′, the values of M' for the points, 1, ... 10, are derived. Table 22 gives the results.

The formula for V_L (page 950), when $\Delta\,w$ is constant, reduces to:

$$V_L = \frac{\sum_B^P x\,x'}{\sum_B^A x^2} = \frac{x_1^2 + x_2^2 + \ldots + x_n^2 - d\,(x_1 + x_2 + \ldots + x_n)}{\sum_B^A x^2}$$

since,

$$x' = x - d \therefore x\,x' = x^2 - d\,x.$$

TABLE 22.

Load at Point: (7)	n. (8)	d. (9)	$n\,d$. (10)	$(x_1 + \ldots + x_n)$. (11)	$(x_1 + \ldots + x_n) - n\,d$. (12)	$M' = \frac{\text{Column (12)}}{20}$. (13)
1′	0	x_1	0.00	0.00	0.00	0.000
2′	1	x_2	37.94	45.80	7.86	0.393
3′	2	x_3	63.12	83.74	20.62	1.031
4′	3	x_4	77.88	115.30	37.42	1.871
5′	4	x_5	84.00	141.26	57.26	2.863
6′	5	x_6	83.20	162.26	79.06	3.953
7′	6	x_7	75.48	178.90	103.42	5.171
8′	7	x_8	61.25	191.48	130.23	6.511
9′	8	x_9	41.20	200.23	159.03	7.951
10′	9	x_{10}	15.30	205.38	190.03	9.504
10	$x_{10} + 9.504 =$					11.204
9	$x_9 + 7.951 =$					13.101
8	$x_8 + 6.511 =$					15.261
7	$x_7 + 5.171 =$					17.751
6	$x_6 + 3.953 =$					20.593
5	$x_5 + 2.863 =$					23.863
4	$x_4 + 1.871 =$					27.831
3	$x_3 + 1.031 =$					32.591
2	$x_2 + 0.393 =$					38.333
1	$x_1 + 0.000 =$					45.800

The right vertical reaction for a load, $P = 1$, at 10′ is $1 - V_L = 1 - 0.4716 = 0.5284$. From symmetry, however, this is the vertical reaction at the left springing line for a load, $P = 1$, at Point 10. The values of V_L for a load, $P = 1$, at Points 10, 9, ... 1, are computed on this principle. Table 23 gives the results.

TABLE 23.

Load at Point. (14)	n. (15)	$x_1^2 + x_2^2 + \ldots + x_n^2$. (16)	$x_1 + x_2 + \ldots + x_n$ (17)	$d = x$. (18)	$d\,(x_1 + \ldots + x_n)$. (19)	Column (16) − Column (19). (20)	$V_L = \frac{\text{Column (20)}}{12\,380}$ (21)
1′	0	0.	0.00	45.80	0.	0.	0.0000
2′	1	2 098.	45.80	37.94	1 736.	362.	0.0292
3′	2	3 537.	83.74	31.56	2 645.	892.	0.0720
4′	3	4 533.	115.30	25.96	2 998.	1 535.	0.1240
5′	4	5 207.	141.26	21.00	2 967.	2 240.	0.1810
6′	5	5 648.	162.26	16.64	2 701.	2 947.	0.2380
7′	6	5 925.	178.90	12.58	2 254.	3 671.	0.2965
8′	7	6 083.	191.48	8.75	1 676.	4 407.	0.3560
9′	8	6 160.	200.23	5.15	1 031.	5 129.	0.4143
10′	9	6 187.	205.38	1.70	349.	5 838.	0.4716
10	$1 - 0.4716 =$						0.5284
9	$1 - 0.4143 =$						0.5857
8	etc.						0.6440
7							0.7035
6							0.7620
5							0.8190
4							0.8760
3							0.9280
2							0.9708
1							1.0000

The formula for the horizontal thrust is (page 998), for $\Delta\, w$ constant and $P = 1$:

$$H = -\frac{\sum_B^P y\, x'}{\sum_B^A y^2}$$

When n (as hitherto) = the number of points, such as 1′, 2′, ..., between B and P:

$$\sum_B^P y\, x' = \Big[y_1 (x_1 - d) + y_2 (x_2 - d) \ldots\ldots + y_n (x_n - d)$$
$$= x_1 y_1 + x_2 y_2 + \ldots + x_n y_n - d (y_1 + y_2 + \ldots + y_n).$$

For the points, 1′, 2′, ..., 10′, considered, x is always positive, but since y may be positive or negative, it is imperative that it should be given its proper sign, as shown in Table 21, from which,

$$\sum_B^A y^2 = 267.5$$

$$H = \frac{-(x_1 y_1 + \ldots + x_n y_n) + d (y_1 + \ldots + y_n)}{\sum_B^A y^2}$$

The results are given in Table 24.

It will be observed that the formulas for M', V_L, and H, are very simple and direct, so that mistakes can be readily avoided with moderate care. The computations take little, if any, more time than the graphical methods, which can be used to check them. It will be noted, further, that the formulas only apply to the right half of the arch. The quantities for the left half are simply derived, as shown from those pertaining to the right half. Similarly, it is necessary to draw the equilibrium polygons only for the right half of the arch.

TABLE 24.

Load at Point. (22)	n. (23)	$x_1 y_1 + \ldots + x_n y_n$. (24)	d. (25)	$y_1 + \ldots y_n$. (26)	$d\,(y_1 + \ldots + y_n)$ (27)	$-$ Column (24) $+$ Column (27). (28)	$H = \frac{\text{Column (28)}}{267.5}$ (29)
1′	0	0.00	45.80	0.00	0.00	0.00	0.000
2′	1	−398.92	37.94	− 8.71	330.46	68.46	0.256
3′	2	−559.41	31.56	−12.94	408.39	151.02	0.565
4′	3	−609.91	25.96	−14·54	377.46	232.45	0.869
5′	4	−607.83	21.00	−14.46	303.66	304.17	1.138
6′	5	−582.42	16.64	−13.25	220.48	361.94	1.353
7′	6	−550.30	12.58	−11.32	142.41	407.89	1.525
8′	7	−519.35	8.75	− 8.86	77.52	441.83	1.652
9′	8	−494.85	5.15	− 6.06	31.21	463.64	1.733
10′	9	−479.40	1.70	− 3.06	5.20	474.20	1.773

To compare results with those found by Mr. Whitney, who made Δx constant in the division of the arch, the values of H, M', and V_L, were plotted for the x's corresponding and the quantities scaled off at the points, 5 ft. apart horizontally, marked 1, 2, 3, ... 10, 9′, ... 2′, 1′, on Fig. 28.

The results are given in Table 25, where Columns (1) refer to the writer's computations and Columns (2) to Mr. Whitney's values.

TABLE 25.

Point.	H. (1)	H. (2)	M' (1)	M' (2)	V_L. (1)	V_L. (2)
1	0.030	0.049	45.06	45.10	0.997	0.994
2	0.195	0.189	40.30	40.25	0.980	0.979
3	0.402	0.396	35.66	35.66	0.953	0.952
4	0.648	0.643	31.25	31.22	0.914	0.914
5	0.920	0.912	27.05	27.05	0.865	0.864
6	1.186	1.174	23.12	23.15	0.806	0.806
7	1.428	1.407	19.46	19.50	0.739	0.738
8	1.613	1.592	16.06	16.14	0.665	0.664
9	1.740	1.712	12.98	13.12	0.583	0.583
10	1.780	1.752	10.33	10.43	0.500	0.500
9′	.....	.....	7.98	8.12	0.417	0.417
8′	.....	.....	6.06	6.15	0.335	0.336
7′	.....	.....	4.46	4.50	0.261	0.262
6′	.....	.....	3.12	3.15	0.194	0.194
5′	.....	.....	2.05	2.08	0.135	0.136
4′	.....	.....	1.25	1.25	0.086	0.086
3′	.....	.....	0.66	0.67	0.047	0.048
2′	.....	.....	0.30	0.29	0.020	0.021
1′	.....	.....	0.06	0.08	0.003	0.006

Columns (1), $\Delta w = \frac{\Delta s}{E I}$ constant.

Columns (2), Δx constant.

On the whole, the quantities agree remarkably well. The differences are greatest with the load, $P = 1$, at the extreme point, 1′, as was naturally expected. The exact values have been determined by the author by integration. Thus, from Tables 15 and 16, and from Figs. 31 and 32 (for $m = 0.339$, $N = 0.186$), it is found, for the load, $P = 1$, at Point 1′, that,

$$M' = 0.068;\ V_L = 0.0051;\ \text{and}\ H = 0.045;$$

also, for $P = 1$ at Point 2, the exact method gives $H = 0.193$. All these quantities are intermediate in value to those corresponding to the two hypotheses, Δw constant and Δx constant, as given in Table 25.

For the usual combination of loads to give maximum moments, whether at the crown, springing line, or at any other point, the discrepancies, by either method, are negligible.

Horizontal Thrust Due to Temperature Changes.—From Equation (23 *a*):

$$H = \frac{t\,\alpha\,l}{\int_B^A y^2 \frac{s}{E I}},$$

in which, all lengths must be expressed in inches.

As l, s, and y, are in feet, and E in pounds per square inch,

$$H = \frac{E\,12\,l\,\alpha\,t}{\sum_B^A (12\,y)^2} \frac{I}{12\,s} = \frac{E\,l\,\alpha\,t}{12 \sum_B^A (y^2)} \frac{I}{12\,s}$$

In dividing the arch axis into ten parts so that $\frac{s}{I}$ should be the same for each

part, using the true value of I, it was found that,

$$s = 0.521 \frac{I}{1\,000} \therefore \frac{I}{12\,s} = 160.$$

Substituting this value and $E = 2\,000\,000$, $l = 100$, $\alpha = 0.000006$, $\sum_B^A y^2 = 267.5$ (from Table 21),

$$H = 59.9\,t$$

and the moment at the crown,

$$59.9\,t \times 3.093 = 185.2\,t$$

Mr. Whitney's values are $H = 58.9\,t$, and the moment at the crown $= 185.0\,t$.

It is very gratifying to find that the two methods practically agree when only ten divisions of the half-arch are used for this arch of 100 ft. span. In this arch, the ratio of the radial thickness of the rib at the springing line to that at the crown is only 1.5. For a ratio of 3, or for a greater span than 200 ft., more than ten divisions of the half-arch should be used, possibly fourteen, to insure practical accuracy.

With only ten points given the arch axis cannot be precisely sketched. A table showing the co-ordinates of twenty points on the axis would give all the accuracy desirable for ordinary spans.

The thrusts from dead and live loads cause rib-shortening, inducing stresses which have too often been only roughly estimated. Thanks to the author more precise methods are now available with diagrams to aid in the computation of such stresses.

The author has recently published an illuminating article* on the special methods adopted by M. E. Freyssinet, the noted French engineer, to reduce shrinkage, rib-shortening, and abutment deflection in concrete spans of nearly 600 ft. Freyssinet's statements regarding the shrinkage effect on actual bridges are quite startling. As Mr. Whitney puts it, "it will be seen that for the ordinary 1 : 2 : 4 mix the shrinkage effect may be as great as the effect of a drop in temperature of from 60 to 90° Fahrenheit."

It is evidently much to be desired that a thorough investigation of this subject should be made, not on blocks of concrete, but on actual reinforced concrete arches. The effect of shrinkage is equivalent to that of a drop in temperature, but the amount has hitherto been taken as much less than Freyssinet's observations would indicate.

Formulas for the arch subjected to horizontal loads have been derived; but they are not quite as simple as those given for vertical loads, as more than the half-arch has to be considered in deriving them. They will not be given since horizontal loads are rarely considered; besides, there is a simple graphical method available.

The demonstration of Maxwell's theorem of reciprocal displacements, as given by the author, is very complete. However, in the application of these theorems, only the case of the arch fixed at the right end and free at the left end is considered. For this case, a simple demonstration will be

* *Engineering News-Record*, September 18, 1924, p. 463.

given here, by aid of the well known Equations (8) and (10). Thus, consider the arch rib of Fig. 59, fixed at the right end, free at the left end, and subjected to a single horizontal load, $P = 1$, at the point, A. The case is quite general since y and y'' may be positive or negative.

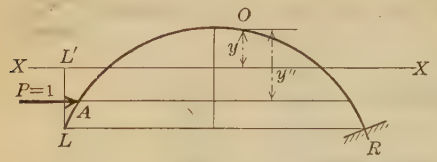

FIG. 59

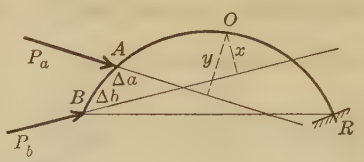

FIG. 60.

Regard L' as the free end; then, for any point, O, y, being always reckoned from the free end, is as shown on Fig. 59. For the load, $P = 1$, the moment, M, about Point O is, $-1\,y'' = -y''$; hence, the horizontal displacement of the free end, L', is:

$$\Delta x = \int_A^R M\,y\,d\,w = -\int_A^R y''\,y\,d\,w$$

Next, consider the load, P, to be removed, and that a single horizontal force, $H = 1$, is applied at L'. This gives at O the moment, $M = -1\,y = -y$. As the horizontal deflection, $\Delta x'$, of Point A is desired, regard Point A as the free end; then, since the ordinates must be reckoned from the free end, y'' takes the place of y in Equation (10), giving the horizontal deflection of A, due to the load, $H = 1$,

$$\Delta x' = \int_A^R M\,y''\,d\,w = -\int_A^R y\,y''\,d\,w$$

since the deformation of the portion of the rib, $R\,A$, alone causes the deflection at A. This being exactly the same expression as was found for Δx, $\Delta x = \Delta x'$; or, the horizontal displacement of L' due to $P = 1$ at A is equal to the horizontal displacement of A caused by the horizontal force, $H = 1$, at L'. This is the first of Maxwell's theorems. The same result follows from replacing the sign of integration by that of summation, for the sum of the products, $y\,y''\,\Delta w$, from A to R, is the same in whatever order the factors are written.

From Fig. 6 and Equation (8), the rotation of the tangent to the neutral line at the free end, L, of the arch rib, Fig. 59, due to the load, $P = 1$, at A, is:

$$\Delta\phi = -\int_A^R M\,d\,w = \int_A^R y''\,d\,w^*$$

Now, suppose that P is removed and that a couple of moment, $M' = 1$, acts at L. Then, regarding A as the free end, the horizontal displacement, Δx, of A is, by Equation (10):

$$\Delta x = \int_A^R M'\,y''\,d\,w = \int_A^R y''\,d\,w,$$

* Referring to Fig. 6 and Equation (8), it may be noted that when M, the moment of the loads on the part of the cantilever from A to (x, y) about (x, y), is negative or counter-clockwise, that $\Delta\phi$ is positive representing an increase in the angle; hence, to agree with this convention as to signs, Equation (8) should be written, $\Delta\phi = -\int_A^B M\,d\,w$. If Equations (9) and (10) are used as written, then, when M is negative, A moves to the right and downward, as in the diagram; when M is positive, A moves upward and to the left.

since y'' is now the ordinate referred to the supposed free end, A. $\therefore \Delta\phi = \Delta x$. This proves the second of Maxwell's theorems for the rib fixed at one end: That the rotation of the tangent at L in radians caused by the force, $P = 1$, at A, is equal to the linear displacement of the point, A, in the direction of P, produced by the couple the moment of which is $M' = 1$ acting at L. The third of Maxwell's theorems is similarly easily proved.

A more general demonstration can be given by supposing two inclined unit forces, P_a at A, and P_b at B, Fig. 60, to act alternately on the arch rib, fixed at the right end and free at the left end.

When P_a alone is acting, suppose the deflection of B in the direction of P_b to be Δb, and when P_b alone is acting, suppose the deflection of A in the direction of action of P_a to be Δa. To prove that $\Delta a = \Delta b$, use will again be made of Equation (10), noting that y is the ordinate of O with reference to the free end and that it is always perpendicular to the direction of the deflection of this end. This likewise can be easily proved directly.

In Fig. 60, let x and y be the perpendiculars from O to P_b and P_a produced, respectively. Thus, x is perpendicular to the direction of Δb and y is perpendicular to the direction of Δa.

When $P_a = 1$ alone acts on the rib at A, then regarding B as the free end, having a deflection, Δb, in the direction of action of P_b, x perpendicular to Δb must replace y in Equation (10). Likewise, $M = -P_a y$. Hence, using summation in place of integration:

$$\Delta b = \sum_A^R M x \Delta w = -\sum_A^R x y \Delta w$$

Next, suppose that P_a is removed and that $P_b = 1$ at B alone acts on the rib, causing the deflection, Δa, at A in the direction of action of P_a. Now, y perpendicular to Δa replaces the y of Equation (10), and $M = -P_b x = -x$. The deformation of the portion of the rib, RA, alone causes the deflection at A, so that the integration will only extend from R to A,

$$\therefore \Delta a = \sum_A^R M y \Delta w = -\sum_A^R x y \Delta w$$

It follows that $\Delta a = \Delta b$, which proves the first of the Maxwell theorems.

In summing from R to A, x will be positive or negative according as O is above or below the line of action of P_b; similarly for y, if the line of action of P_a produced passes above R.

Now consider the rotation, $\Delta\phi$, of the tangent to the neutral line at B when $P_a = 1$ alone acts. This rotation is due to the deformations in the rib from R to A and is the same at B as at A. Then $M = -P_a y = -y$, and by Equation (8):

$$\Delta\phi \text{ (at } B) = -\sum_A^R M \Delta w = \sum_A^R y \Delta w$$

Next, suppose P_a to be removed and that P_b at B is replaced by a couple the moment of which is $M' = 1$. This couple, acting alone on the rib, pro-

duces a deflection, Δa, at A in the direction of action of P_a. Taking A as the free end, as before,

$$\Delta a = \sum_A^R M y \Delta w = \sum_A^R y \Delta w$$

$$\therefore \Delta a = \Delta \phi$$

giving the second of Maxwell's theorems. The third theorem follows, because for the deformations in $R A$ only, $\Delta \phi$ at A and B must be the same.

As far as the writer knows, the preceding demonstrations are new.

In conclusion, the writer thinks that the author has given a very valuable paper—one that has added materially to present knowledge of arch bridges and that should lead to improved designs.

CARL B. ANDREWS,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's proposal to make I inversely proportional to $\cos \phi$ is a departure from the previous general rule that the radial thickness of the arch rib should be inversely proportional to $\cos \phi$. The old rule would make the vertical projection of all radial sections equal; the author's suggestion would result in the third power of the radial thickness being inversely proportional to $\cos \phi$. In the case of arches with considerable rise, the ribs might be rather thin.

It is evident that the author has done an immense amount of work in the preparation of this paper, for which he deserves the thanks of all engineers interested in arch design. The formidable appearance of his demonstrations, however, is somewhat against the general adoption of his method. Engineers are prone to think that a single method of arch analysis is enough for one man to master in one life time, and are slow to take up a new method which looks just as hard as the old one to which they are accustomed.

R. R. MARTEL,† ASSOC. M. AM. SOC. C. E. (by letter).—The use of the influence lines for arches given in the latter part of this thorough and painstaking paper will prove a great aid in determining the preliminary shape of arch axes and the economical cross-sections at the crown and springing lines; but with large spans, especially where the shape of the arch axis and the variation in the cross-sections throughout do not conform closely with the data assumed in obtaining these influence lines, it will still be desirable and in some cases necessary to make analyses of stresses, using the actual data. Opinions regarding the simplest and best method of doing this will be influenced, to a large degree, by the familiarity of the user with the method he sponsors.

To those who are not already committed to one of the older processes of analysis the method presented here may appeal. It has the advantage of avoiding the use of the funicular polygons, being comparatively simple and largely mechanical. Instead of dividing the arch axis so as to make $\frac{d s}{I}$ a constant, or of using arbitrary finite divisions of the arch axis and replacing the integrals in the expressions for crown thrust, moment, and shear, with

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summation signs, the values of the functions involved are plotted to scale and integration is performed by quadrature; that is, by finding the area under the curves of these functions by planimeter or similar means.

Part I.—Arch Analysis with Planimeter

The equations for the crown thrust, shear, and moment, in an elastic arch under vertical loading, given herewith are readily understood and will not be derived here.

$$H_0 = \frac{\int \frac{d\,s}{I} \int \frac{m\,y\,d\,s}{I} - \int \frac{m\,d\,s}{I} \frac{y\,d\,s}{I}}{2\left[\int \frac{d\,s}{I} \int \frac{y^2\,d\,s}{I} - \left(\int \frac{y\,d\,s}{I}\right)^2\right]}$$

$$H' = \frac{\Delta \int \frac{d\,s}{I}}{2\left[\int \frac{d\,s}{I} \int \frac{y^2\,d\,s}{I} - \left(\int \frac{y\,d\,s}{I}\right)^2\right]}$$

$$V_0 = \frac{\int \frac{(m_L - m_R)\,x\,d\,s}{I}}{2\int \frac{x^2\,d\,s}{I}}$$

$$M_0 = \frac{\int \frac{m\,d\,s}{I} - 2\,H_0 \int \frac{y\,d\,s}{I}}{2\int \frac{d\,s}{I}}$$

$$M' = \frac{-H' \int \frac{y\,d\,s}{I}}{\int \frac{d\,s}{I}}$$

For a symmetrical arch the limits of all the integrals are for half the arch.

In these equations:

ds = differential length of the arch axis;

I = moment of inertia of any section (I of concrete area $+ \frac{E_s}{E_c} \times I$ of steel area);

x, y = co-ordinates of any point on the arch axis with the crown as origin, and all taken as positive;

l = length of span;

E_c = modulus of elasticity of concrete;

E_s = modulus of elasticity of steel;

m = bending moment at any point due to external loads considering each half of the arch as a cantilever. Where m occurs without subscripts, it includes the bending moments at symmetrical points on both halves of the arch. In all cases, m has been taken positive for downward loads;

$\Delta = c\ t\ l\ E_c$ in the case of temperature change;
$= p\ l$ in the case of rib-shortening due to thrust;
$= k\ l\ E_c$ in the case of shrinkage;
c = coefficient of expansion;
t = temperature rise or fall, taken as positive for a rise;
p = average intensity of compression on equivalent concrete area

$$\left(\text{area of concrete} + \frac{E_s}{E_c} \times \text{area of steel}\right);$$

k = coefficient for shrinkage;
H_o = thrust at the crown, positive for compression;
V_o = shear at the crown, positive when acting as in Fig. 61(a);
M_o = moment at the crown, positive when causing compression in the top;
H' = thrust at crown due to temperature change or rib-shortening;
M' = moment at crown due to rib-shortening or temperature change;
α = angle between tangent to arch axis at any point and the horizontal.

As $ds = \frac{dx}{\cos \alpha}$, the term, $\frac{ds}{I}$, in the equations may be replaced by $\frac{dx}{I'}$, in which $I' = I \cos \alpha$. Note that α is also the angle which the radius of curvature at any point makes with the vertical.

As influence values are sought, the value of m due to a unit load may be replaced by $(x - n\,a)$ for values of x greater than $n\,a$. For values of x less than $n\,a$, $m = 0$. Considering the left half of the arch,

$$(m_L - m_R) = (x - n\,a),$$

as $m_R = 0$.

Procedure.—The arch is first divided into any convenient number, n, of equal horizontal divisions each a in length. It is not necessary that a should be an aliquot part of the span, but it is somewhat simpler if a is a number of even feet in length. When a can be made 10 ft., some of the data can be reduced by shifting the decimal point.

Values of $\frac{1}{I'}$, $\frac{y}{I'}$, and $\frac{y^2}{I'}$ are computed for each of the n sections. These are all the preliminary computations needed. The steps in the analysis then are:

(1a).—Plot the values of $\frac{1}{I'}$ as ordinates with corresponding values of n as abscissas and connect these points with a smooth curve, A, Fig. 61 (b). The area of a strip, $p\,q$, is then $\frac{dx}{I'}$, and the area between Curve A and the axis of ordinates is equal to $\int \frac{dx}{I'}$.

(1b).—Plot n times the value of $\frac{1}{I'}$ on the same base at each corresponding value of n and connect the points thus found with a smooth curve, B, Fig.

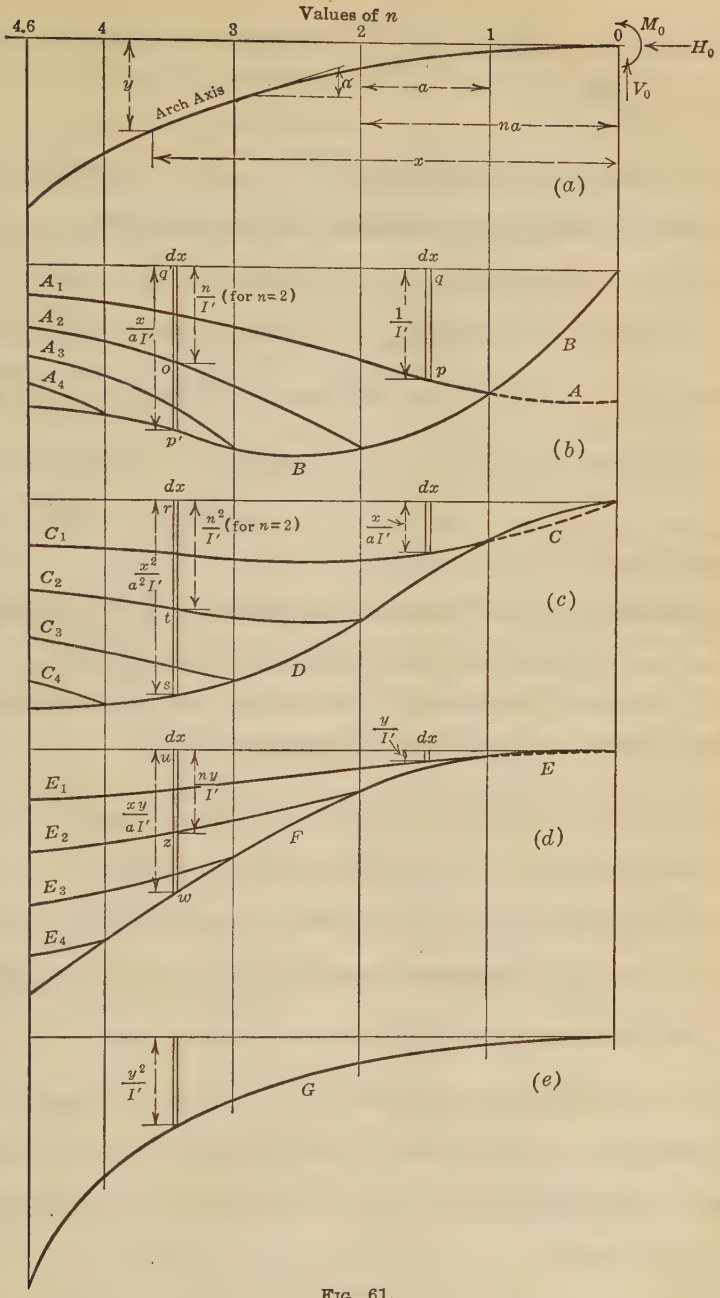
Values of n
4.6
4
3
2
1
0
M_0
H_0
V_0
y
Arch Axis
α
a
na
x
(a)
dx
A_1
A_2
A_3
A_4
q
$\frac{n}{I'}$ (for n=2)
$\frac{x}{aI'}$
o
p'
$\frac{1}{I'}$
p
B
A
(b)
C_1
C_2
C_3
C_4
r
$\frac{n^2}{I'}$ (for n=2)
$\frac{x^2}{a^2I'}$
t
s
$\frac{x}{aI'}$
C
D
(c)
E_1
E_2
E_3
E_4
u
$\frac{ny}{I'}$
$\frac{xy}{aI'}$
z
w
$\frac{y}{I'}$
E
F
(d)
$\frac{y^2}{I'}$
G
(e)

FIG. 61.

61 (*b*). Since the ordinates of Curve *B* are values of $\frac{n}{I'}$, the area of a strip, $p'\ q'$, is $\frac{n\,dx}{I'}$, and as $n\,a = x$, this area is $\frac{x\,dx}{a\,I'}$. Thus, the area between Curve *B* and the axis is $\frac{1}{a}\int \frac{x\,dx}{I'}$.

(1c).—Connect all points representing $\frac{2}{I'}$, $\frac{3}{I'}$, $\frac{4}{I'}$, etc., beginning at Curve *B* and extending to the left to the springing line, giving Curves A_1 to A_4, Fig. 61 (*b*).

Assume a unit load placed at $n = 2$; the value of m at any point, x, is 1 $(x - n\,a)$ which may be written $a\left(\frac{x}{a} - n\right)$, and the value of $\frac{m\,dx}{I'}$ may be expressed as $a\left(\frac{x}{a} - n\right)\frac{dx}{I'}$. Since the length of the strip, $o\,q'$, is $\frac{2}{I'}$ and of $p'\ q'$, $\frac{x}{a\,I'}$, the length of $p'\ o$ is $\left(\frac{x}{a} - n\right)\frac{1}{I'}$, and the area between Curve *B* and Curve A_2 is $\frac{1}{a}\int \frac{m\,dx}{I'}$ for a unit load at $n = 2$, the limits being from $n = 2$ to the springing line.

For a unit load at any other value of n, the value of $\frac{1}{a}\int \frac{m\,dx}{I'}$ may be found in the same way by evaluating the area between Curve *B* and Curve A_n.

(2*a*).—With a new base, Fig. 61 (*c*), plot Curve *B* using a smaller scale for the ordinates, which will locate Curve *C*, giving values of $\frac{n}{I'}$.

(2*b*).—Plot n times these values of $\frac{n}{I'}$ at each corresponding value of n and connect the points thus found with a smooth curve, *D*, Fig. 61 (*c*).

Since the ordinates of Curve *D* are values of $\frac{n^2}{I'}$, the area of a strip, $r\ s$, is $n^2\frac{d\,x}{I'}$, and as $n\,a = x$, this area is equal to $x^2\frac{d\,x}{a^2\,I'}$. Thus, the area between Curve *D* and the axis is $\frac{1}{a^2}\int x^2\frac{d\,x}{I'}$.

(2c).—Connect all points representing $\frac{2\,n}{I'}$, $\frac{3\,n}{I'}$, $\frac{4\,n}{I'}$, etc., beginning at Curve *D* and extending to the left to the springing line, giving Curves C_1 to C_4, Fig. 61 (*c*).

Assume a unit load placed at $n = 2$. As in Item (1c), the value of m is $a\left(\frac{x}{a} - n\right)$. Hence,

$$m\,x = a\,x\left(\frac{x}{a} - n\right) = a^2\left(\frac{x^2}{a^2} - n^2\right)$$

and,

$$m\,x\,\frac{d\,x}{I} = a^2\left(\frac{x^2}{a^2} - n^2\right)\frac{d\,x}{I}$$

$x^2\,\frac{d\,x}{a^2\,I}$ is the area of the strip, $r\,s$, and $n^2\,\frac{d\,x}{I}$ is the area of the strip, $r\,t$, when $n = 2$. Hence, the area of the strip, $t\,s$, is:

$$\left(\frac{1}{a^2}\right)\left(m\,x\,\frac{d\,x}{I}\right),$$

and the area between Curve D and Curve C is equal to:

$$\left(\frac{1}{a^2}\right)\int m\,x\,\frac{d\,x}{I}$$

the limits being from $n = 2$ to the springing line.

For a unit load at any other value of n, the value of:

$$\left(\frac{1}{a^2}\right)\int m\,x\,\frac{d\,x}{I}$$

may be found in the same way by evaluating the area between Curve D and Curve C_n.

(3a).—Plot the values of $\frac{y}{I}$ as ordinates from a third base, using the corresponding values of n as abscissas, and connect these points with a smooth curve, E, in Fig. 61 (d). The area between this curve and the axis of ordinates is equal to $\int\frac{y\,d\,x}{I}$.

(3b).—Plot n times the value of $\frac{y}{I}$ on the same base at each corresponding value of n and connect the points thus found with a smooth curve, F, Fig. 61 (d).

(3c).—Connect all points representing $\frac{2\,y}{I}$, $\frac{3\,y}{I}$, $\frac{4\,y}{I}$, etc., beginning at Curve F and extending to the left to the springing line, giving Curves E_1 to E_4, Fig. 61 (d).

Assume a unit load at $n = 2$. The value of $m\,y$ is then:

$$y\,(x - n\,a), \text{ or } a\,y\left(\frac{x}{a} - n\right)$$

and,

$$m\,y\,\frac{d\,x}{I} = a\,y\left(\frac{x}{a} - n\right)\frac{d\,x}{I}$$

The area of the strip, $u\,w$, is $\left(\frac{x\,y}{a\,I}\right)d\,x$, and that of the strip, $u\,z$, is $n\,y\,\frac{d\,x}{I}$. Hence, the area of the strip, $z\,w$, is $\frac{1}{a}\left(m\,y\,\frac{d\,x}{I}\right)$ and the area between Curves F and $E_2 = \frac{1}{a}\int m\,y\,\frac{d\,x}{I}$ for a unit load at $n = 2$.

For a unit load at any other value of n, the value of $\frac{1}{a}\int m\,y\,\frac{d\,x}{I'}$ may be found in the same way by evaluating the area between Curve F and Cur ve E_n.

(4).—Plot the values of $\frac{y^2}{I'}$ as ordinates on a fourth base, Fig. 61 (*e*). The area between this curve and its axis of ordinates is the $\int y^2\frac{d\,x}{I'}$.

(5).—The values obtained by evaluating the various areas in Items (1) to (4) may now be entered in a table similar to Table 26. The remainder of the work of obtaining the influence values for H_0, V_0, and M_0 is indicated at the top of the columns of this table, the figures in parentheses referring to the steps in the foregoing outline.

(6).—Computing Δ involves no difficulty, and as all the other terms in the equation for H' have already been found, the crown thrust and moment due to temperature or rib-shortening are readily obtained by substituting in the expressions for these quantities.

TABLE 26.—Computations in Solving for H_0, V_0, and M_0 for Influence Values.

n	(2*c*)	(2*b*) V_0	(3*c*)	(1*c*)	(1*a*)	(3*a*)		H_0				M_0
	$\frac{1}{a^2}\int\frac{m\,x\,d\,x}{I'}$	$\frac{\text{Column (1)}}{2\times 116.7}$	$\frac{1}{a}\int\frac{m\,y\,d\,x}{I'}$	$\frac{1}{a}\int\frac{m\,d\,x}{I'}$	13.3 a × Column (3)	55.2 a × Column (4)	Column (5) − Column (6)	$\frac{\text{Column (7)}}{10\,520}$	a × Column (4)	55.2 × 2 H_0	Column (9) − Column (10)	$\frac{\text{Column (11)}}{2\times 13.3}$
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0	116.7	0.500	252	31.4	33 500	17 300	16 200	1.54	314	170	144	−5.41
1	71.5	0.320	167	15.4	22 200	8 500	13 700	1.30	154	143	11	−0.41
2	36.1	0.161	960	6.1	12 750	3 360	9 390	0.89	61	98	−37	−1.39
3	12.0	0.054	400	1.6	5 320	880	4 440	0.42	16	47	−31	−1.17
4	1.8	0.008	80	0.2	1 060	110	950	0.09	2	10	− 8	−0.30
4.6	0.0	0.000	0	0.0	0	0	0	0.00	0	0	0	0.00

Notes.—

$a = 10$

Column (2): $\frac{1}{a^2}\int\frac{x^2\,d\,x}{I'} = 116.7$; Column (5): $\int\frac{d\,x}{I'} = 13.3$

Column (6): $\int\frac{y\,d\,x}{I'} = 55.2$; From Fig. 61 (*e*): $\int\frac{y^2\,d\,x}{I'} = 624$

Column (8): $2\left[\int\frac{y^2\,d\,x}{I'}\int\frac{d\,x}{I'} - \left(\int\frac{y\,d\,x}{I'}\right)^2\right] = 2\left[624\times 13.3 - (55.2)^2\right] = 10\,520$

Part II.—Arch Analysis with Integraph

A modification of the method outlined in Part I which will simplify the work still further can be applied when an integraph is available. In the

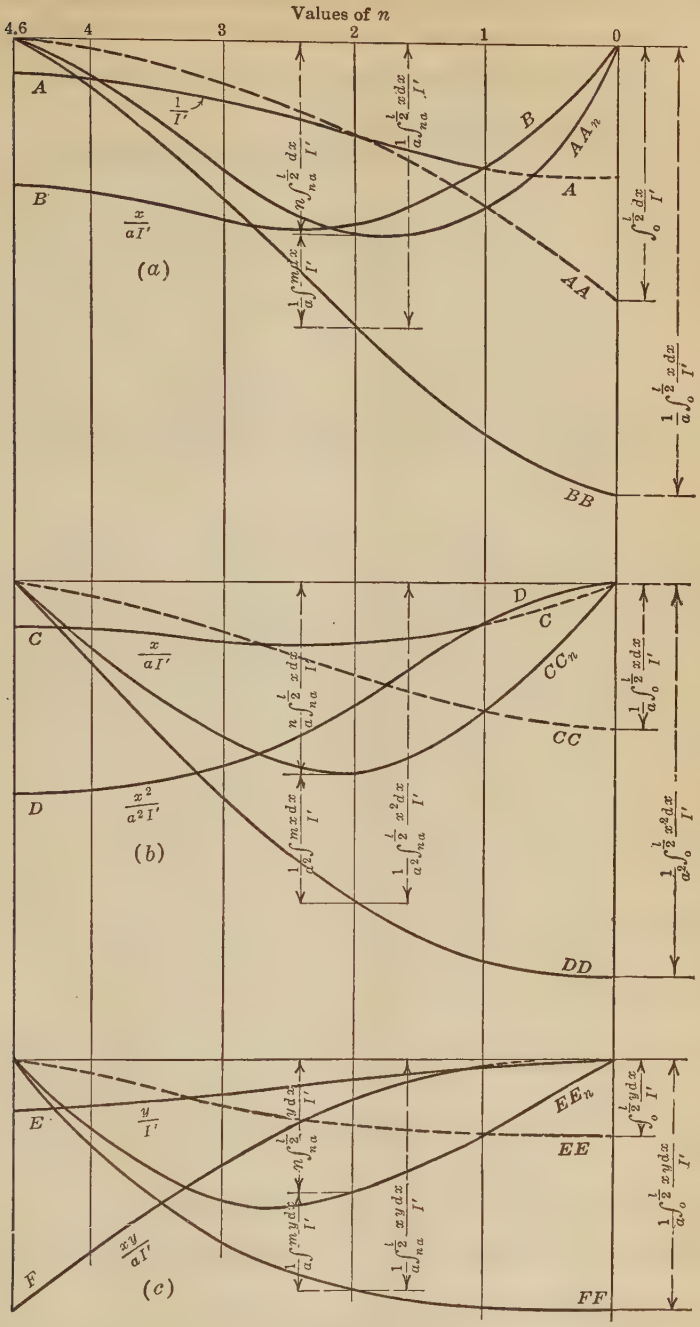
Values of n
4.6
4
3
2
1
0
A
B
$\frac{1}{I'}$
$\frac{x}{aI'}$
(a)
$\frac{1}{a}\int \frac{mx\,dx}{I'}$
$n\int_{na}^{\frac{1}{2}} \frac{dx}{I'}$
$\frac{1}{a}\int_{na}^{\frac{1}{2}} \frac{x\,dx}{I'}$
$\int_{0}^{\frac{1}{2}} \frac{dx}{I'}$
$\frac{1}{a}\int_{0}^{\frac{1}{2}} \frac{x\,dx}{I'}$
AA_n
AA
BB
C
D
$\frac{x^2}{a^2I'}$
(b)
$\frac{1}{a^2}\int \frac{mx^2dx}{I'}$
$n\frac{1}{a}\int_{na}^{\frac{1}{2}} \frac{x\,dx}{I'}$
$\frac{1}{a^2}\int_{na}^{\frac{1}{2}} \frac{x^2dx}{I'}$
$\frac{1}{a}\int_{0}^{\frac{1}{2}} \frac{x\,dx}{I'}$
$\frac{1}{a^2}\int_{0}^{\frac{1}{2}} \frac{x^2dx}{I'}$
CC_n
CC
DD
E
F
$\frac{y}{I'}$
$\frac{xy}{aI'}$
(c)
$\frac{1}{a}\int \frac{myx\,dx}{I'}$
$n\frac{1}{a}\int_{na}^{\frac{1}{2}} \frac{yx\,dx}{I'}$
$\frac{1}{a}\int_{na}^{\frac{1}{2}} \frac{xy\,dx}{I'}$
$\int_{0}^{\frac{1}{2}} \frac{xy\,dx}{I'}$
$\frac{1}{a}\int_{0}^{\frac{1}{2}} \frac{xy\,dx}{I'}$
EE_n
EE
FF

FIG. 62.

fundamental equations given at the beginning of Part I, the value of m for a unit load at a distance, $n\,a$, from the origin is $x - n\,a$, and the integrals in which m occurs are integrated between the limits, $n\,a$ and $\frac{l}{2}$ (since for values of x less than $n\,a$, m vanishes).

Then the various integrals involving m in the following forms may be written:

$$\int \frac{m\,d\,s}{I} = \int \frac{x\,d\,x}{I'} - n\,a \int \frac{d\,x}{I'} = a\left[\int \frac{x\,d\,x}{a\,I'} - n \int \frac{d\,x}{I'}\right] \ldots (65)$$

$$\int \frac{m\,x\,d\,s}{I} = \int \frac{x^2\,d\,x}{I'} - n\,a \int \frac{x\,d\,x}{I'} = a^2\left[\int \frac{x^2\,d\,x}{a^2\,I'} - n \int \frac{x\,d\,x}{a\,I'}\right] . (66)$$

$$\int \frac{m\,y\,d\,s}{I} = \int \frac{x\,y\,d\,x}{I'} - n\,a \int \frac{y\,d\,x}{I'} = a\left[\int \frac{x\,y\,d\,x}{a\,I'} - n \int \frac{y\,d\,x}{I'}\right] \ldots (67)$$

The last two terms in each of Equations (65), (66), and (67) are designated as B, A; D, C; and F, E, respectively, on Fig. 62. The limits for all the integrals are $\frac{l}{2}$ and $n\,a$.

Curves for all the expressions under the integrals on the right of the equations have been constructed in Part I. With the integraph the integral curves of all these expressions are drawn beginning at the springing line, and these integral curves are designated with the double letters in Fig. 62.

In a manner similar to that used in Part I, plot n times the values of the ordinates to Curves $A\,A$, $C\,C$, and $E\,E$, at the corresponding values of n, giving Curves $A\,A\,n$, $C\,C\,n$, and $E\,E\,n$, the ordinates of which would be equal to: $n \int \frac{d\,x}{I'}$, $n \int \frac{x\,dx}{a\,I'}$, and $n \int \frac{y\,d\,x}{I'}$, respectively, these being the last terms in each of the three Equations (65), (66), and (67).

Then the vertical intercept between Curves $A\,A\,n$ and $B\,B = \frac{1}{a} \int \frac{m\,d\,s}{I}$; the vertical intercept between Curves $C\,C\,n$ and $D\,D = \frac{1}{a^2} \int \frac{m\,x\,d\,s}{I}$; and the vertical intercept between Curves $E\,E\,n$ and $F\,F = \frac{1}{a} \int \frac{m\,y\,d\,s}{I}$.

The ordinates at the crown, $n = 0$, to Curves $B\,B$ and $D\,D$ are $\frac{1}{a} \int_0^{\frac{l}{2}} \frac{x\,d\,x}{I'}$ and $\frac{1}{a^2} \int_0^{\frac{l}{2}} \frac{x^2\,d\,x}{I'}$, respectively. The ordinates at the crown, $n = 0$, to Curves $A\,A$ and $E\,E$ are $\int_0^{\frac{l}{2}} \frac{d\,x}{I'}$ and $\int_0^{\frac{l}{2}} \frac{y\,d\,x}{I'}$, respectively.

Therefore, all the integrals required in the fundamental equations are now found except $\int_0^{\frac{l}{2}} \frac{y^2\,d\,x}{I'}$, for which it is simpler to find the area under Curve G of Fig. 61 (e) with a planimeter than to construct an integral curve.

The remainder of the work of obtaining influence values for H_0, V_0, and M_0 can best be done in tabular form as in Part I.

E. H. HARDER,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer has used the method and tables of Strassner for several years with considerable success and is glad to see this praiseworthy work translated into English. If a hingeless symmetrical concrete arch is designed according to the shape and form computed by this method, its tables or curves are accurate and can be used with confidence; but the writer disagrees with the author's statements that they can be used without a thorough knowledge of the entire theory of indeterminate arches. The writer is quite certain that Strassner never intended his tables to be used by a novice. They are invaluable, however, to a mature designer who has designed such arches according to more laborious methods and who may then expect to have his work made a little pleasanter. For ordinary earth-filled arches, these tables may be readily used, but for large spans with transverse spandrel walls or columns, the novice could not be expected to know their limitations or how to make the proper allowances.

Beginning with Section IV, "The Arch Axis" (page 966), the author gives the mathematical derivation of the equation for a curve known as a transformed catenary. This curve is a linear arch for a load of uniform density between a horizontal roadway and the curve itself. The linear arch for a uniform horizontal load is, of course, a parabola. For filled arches with a horizontal roadway, the transformed catenary is the correct curve to use, but for large open spandrel arches a departure from this curve is often necessary.

Judging from a number of textbooks which have been written, one gains the impression that any reasonable curve (segment of a circle, parabola, or ellipse) is the correct curve for the arch axis provided one later blindly applies the elastic theory to determine the moments and thrusts. For live loads, this procedure is not questioned, but for dead loads, it is inaccurate and decidedly questionable in the minds of some engineers.

The dead load pressure line for any well designed concrete arch should coincide exactly with the center line of the arch. For earth-filled arches with a horizontal roadway such a dead load pressure line is a transformed catenary. For other conditions of loading the shape of the arch center line may be slightly different from such a catenary. If this condition has been complied with, then, neglecting for a moment arch-shortening, there will be uniform stress over every radial section passed through the arch. Due, however, to the elastic properties of the material of the arch, moments of a secondary nature will be caused by what is known as arch-shortening, which is clearly treated in the author's text. Any other shape for the arch center line will cause bending moments throughout the arch in addition to those caused by arch-shortening. There are reasons to believe that such moments are questionable; whether questionable or not, they may seriously impair the economy of the arch.

Some writers (Tolkmit, Melan, and others) advocate a transformed catenary designed for the dead loads plus one-half the uniform live load over the

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entire span. Such a curve can be computed by the method given in the paper by adding $\frac{1}{2}\,p$ to w_s and w_c, respectively. The writer, however, prefers the first method.

To eliminate the secondary moments due to arch-shortening has been the aim of engineers since the beginning of the construction of modern concrete arches. In recent years several writers have sought to accomplish this by a further modification of the transformed catenary or "line-of-thrust" arch, that is, by a change of curvature. In general, such a change means a flatter curve at the crown and a sharper curve at the springing line. Two interesting papers on this phase of the subject have been written by Gerhard Neumann* and Professor A. Ostenfeld,† of Copenhagen, Denmark. The latter paper is mathematical jugglery which gives excellent results on paper, but the writer has been unable to conceive how a secondary moment can be overcome by a mere change of shape. Arch-shortening thrusts are produced by an elastic deflection and such thrusts can only be eliminated or neutralized by external forces or moments producing similar deflections in the reverse direction. In the first edition of his book, Strassner‡ also gives a method for improving the shape of the arch axis but, in a later edition, this "improvement" is omitted.

The earliest recognized methods of eliminating the dead load arch-shortening moments called for temporary hinges at the crown and springing lines of the arch. After this three-hinged arch carried all the dead loads, the hinges were closed and thereafter the arch acted as a hingeless structure for live loads, temperature changes, shrinkage, and additional abutment displacements. This method is practical but expensive, especially for long heavy ribs. Few typical examples of such construction methods are recorded. In 1908, the Stubenrauchbrücke was built in Berlin, Germany. The central span was a 200-ft. steel arch flanked on each end by 64-ft. concrete arches. These concrete arches had temporary hinges at the crowns and springing lines and after the entire dead load of all three spans had been placed, the hinges of the concrete arch were closed. In 1921, a ribbed arch of 135 ft. span with temporary hinges was built in Warrington, England.§

A newer method developed by Buchheim and Heister in Germany and by Freyssinet in France makes use of hydraulic jacks inserted between two planes which cut the crown of the arch. In this manner a horizontal thrust is created which forces the two crown sections apart and lifts the arch from its centering. At the same time, careful measurements are made from which it is possible to determine accurately the modulus of elasticity of the entire structure. This modulus is necessary in order to make the proper allowances for arch-shortening, temperature, shrinkage, and abutment displacements. After the centering has been removed, the pressure on the hydraulic jacks is

* *Beton und Eisen*, No. XIII (1922).
† *Beton und Eisen*, No. XIII (1923).
‡ "Neuere Methoden zur Statik der Rahmentragwerke und der elastischen Bogentraeger."
§ *Le Genie Civil*. July 30, 1921.

reduced to the predetermined amount and the crown section is closed. Although this method ("Gewölbe Expansions Verfahren") was first developed to repair arch bridges which had suffered damage from abutment displacements,* it has been used in the erection of arches in Germany† and France.‡ In the latter country, this method has been developed to such an extent by Freyssinet as to make the economical construction of long-span bridges possible.

Besides eliminating the distressing dead load arch-shortening moments, it also eliminates the uncertainties of the location of the dead load pressure line. Closure of the crown is made at a definite temperature and thereby another indeterminate quantity in the present methods of constructing reinforced concrete arches is removed. Without the use of this method, it is impossible to know the temperature which creates no moments in the structure.

In Section V, "Form of Arch Rib",§ is given the equation of a special case of the law of variation of the moments of inertia throughout the arch rib. The general equation is,

$$\frac{I_c}{I_x \cos \phi} = 1 - (1 - m)\, Z^v$$

in which, the exponent, v, may vary from 0 to 2. In his work, Strassner has chosen v equal to 1 purely for the sake of simplifying those integrations which involve the variable arch moments of inertia.

Little has been written about the subject of the variation of the moments of inertia along the arch axis and wherever it has appeared, it has been treated very superficially. Generally, arbitrary assumptions were made (as, for example, that $I_x \cos \phi$ was constant) or an arbitrary set of proportions was given for the arch depths which proportions were dependent on the depths at the crown and at the haunch. The general law just given was the result of a research which had for its purpose the distribution of the arch moments of inertia so that the maximum stresses would be equal for all sections along the arch axis. If such an arch could be designed, it would be more economical, of course, than an arch of haphazard form wherein the crown stresses might be twice the haunch stresses, or *vice versa.*

In Fig. 63 (*a*) are plotted the maximum positive and negative live load moments of a large hingeless reinforced concrete arch designed according to this method. This curve is typical of all hingeless arches and shows that besides the crown section, there is another section of maximum moments at seven-tenths of the length of the half span measured from the crown. In Fig. 63 (*b*) is a graph of the dead load arch-shortening moments, in Fig. 63 (*c*), a graph of moments caused by temperature changes, and in Fig. 63 (*d*), a graph showing the combination of all moments of Fig. 63 (*a*), (*b*), and (*c*). In Fig. 63 (*d*) are also shown two moment curves which are proportional to the moments of inertia of sections along the arch axis. Curve *a* was computed

* "Handbuch für Eisenbeton", Vol. VII.

† "Handbuch für Eisenbeton", Vol. I.

‡ *Engineering News-Record,* September 18, 1924.

§ See p. 977.

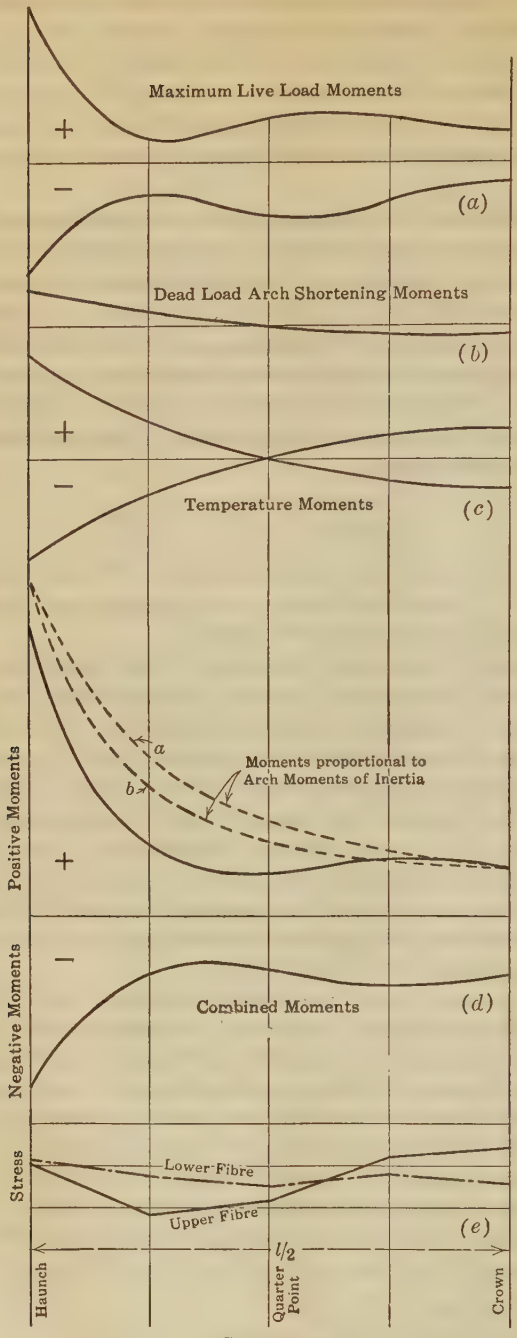
Maximum Live Load Moments
+
−
(a)
Dead Load Arch Shortening Moments
(b)
+
−
Temperature Moments
(c)
a
b
Moments proportional to
Arch Moments of Inertia
Positive Moments
+
Negative Moments
−
Combined Moments
(d)
Stress
Lower Fibre
Upper Fibre
(e)
l/2
Haunch
Quarter Point
Crown

FIG. 63.

according to the method given in the paper and Curve *b* according to the general law with $v = 2$.

Fig. 63 (*e*) is a graph of the maximum fiber stresses for dead and live load, arch-shortening, and temperature changes. The fiber stresses at the haunch are quite close to the maximum crown stresses, but in the region of the minimum moments shown in Fig. 63 (*d*), they are below the allowable maximum. If an arch were designed so that the stresses would neither be greater nor less than the allowable maximum stress, there would result a curious structure which would increase in thickness from the crown to about four-tenths the half span, then decrease to a minimum thickness at about seven-tenths the half span and from that point the arch haunch would have a rapid increase in thickness to the maximum section needed at that point. Such an arch would have an ungainly appearance, and would be unacceptable from an æsthetic point of view no matter what its advantages might be.

The author's method for determining the thickness of arch rib* is very unsatisfactory and is undoubtedly based on the first edition of Strassner's book. In a later edition, he has greatly improved the chapter on the planning or designing of the arch rib.

Equation (46) is that of the dead load horizontal thrust and may be written:

$$H_d = C_d \frac{w_c\, l^2}{r}$$

in which,

$$C_d = \frac{g - 1}{4\, K^2}$$

(see Table 9).

The empiric equation for C_d is given by Strassner:

$$C_d = 0.1080 + 0.0190\, g - 0.0005\, g^2$$

For uniform live loads:

$$H_l = C_l \frac{p\, l^2}{r}$$

in which, for $m = 0.3$:

$$C_l = 0.0579 + 0.0035\, g$$

and

$$M_l = K_l\, pl^2$$

in which, for $m = 0.3$:

$$K_l = 0.00426 + 0.00037\, g$$

These coefficients may be taken from Figs. 50 to 55, but may also be expressed by empiric equations. Strassner gives a list of such equations for various values of m, but as the value of $m = 0.3$ is most prevalent, the writer will use only those equations for this value to derive general formulas for crown and haunch thicknesses:

$$\cos \phi_s = \frac{1}{\sqrt{1 + \tan^2 \phi_s}} = \frac{1}{\sqrt{1 + x \left(\frac{r}{l}\right)^2}}$$

* See p. 977.

in which,

$$x = 11.50 + 4.55\,g - 0.13\,g^2$$

For a change of temperature:

$$H_t = \frac{t\,\alpha\,l}{\int y^2\,d\,w} = \frac{t\,\alpha\,E\,I_c}{C\,r^2}$$

in which, $I_c = \frac{b\,d_c^3}{12}$ for a rectangular cross-section. Then, for $b = 1.0$:

$$H_t = \frac{E\,\alpha\,t\,d^3}{12\,C\,r^2} = C_t\,\frac{t\,d_c^3}{r^2}$$

$$M_t = -\,H_t \cdot y = \frac{E\,\alpha\,t\,d_c^3}{12\,C\,r} \times \frac{y}{r} = K_t\,\frac{t\,d_c^3}{r}$$

C may be obtained from Table 18 and $\frac{y}{r}$ from Table 6.

For $m = 0.3$, these coefficients may be expressed empirically:

$$C_t = 4218 + 136.8\,g$$
$$K_t = -1037.1 + 4.56\,g$$

Dead load rib-shortening is expressed by:

$$H_{RS} = -\,u'\,H_a$$

in which,

$$u' = \frac{l}{E\,A_m'\int y^2\,d\,w}$$

Proceeding as before:

$$H_{RS} = C_e\,\frac{w_c\,l^2\,d_c^2}{r^3}$$

and the section moment is:

$$M_{RS} = K_e\,\frac{w_c\,l^2\,d_c^2}{r^2}$$

in which,

$$C_e = \frac{C_a}{12\,C\,C_m'} \text{ and } K_e = C_e\,\frac{y}{r}$$

For $m = 0.3$:

$$C_e = -0.155 - 0.033\,g$$
$$K_e = 0.0408 + 0.0046\,g$$

The fiber stresses may be computed by the usual formula for homogeneous sections without reinforcement:

$$f_c = \frac{T}{b\,d} \pm \frac{6\,M}{b\,d^2}$$

Then, for a rib, 1 ft. wide:

$$f_c = \frac{T}{d} \pm \frac{6\,M}{d^2}$$

Substituting the terms derived previously:

$$T \cdot \cos\phi = C_a\,\frac{w_c\,l^2}{r} + C_l\,\frac{p\,l^2}{r} + C_t\,\frac{t\,d_c^3}{r^2} + C_e\,\frac{w_c\,l^2\,d_c^2}{r^3}$$

and

$$M = K_l \cdot p\, l^2 + K_t \frac{t\, d_c^3}{r} + K_e \frac{w_c\, l^2\, d_c^2}{r^2}$$

Substituting these equations into the equation for f_c for the crown section:

$$f_c = C_1 \frac{l^2}{d_c\, r} + C_2 \frac{d_c^2}{r^2} + C_3 \frac{l^2\, d_c}{r^3} \pm 6 \left(K_1 \frac{l^2}{d_c^2} + K_2 \frac{d_c}{r} + K_3 \frac{l^2}{r^2} \right)$$

in which,

$$C_1 = C_d \cdot w_c + C_l \cdot p\,; \quad C_2 = C_t \cdot t\,; \quad C_3 = C_e \cdot w_c$$
$$K_1 = K_l \cdot p\,; \quad K_2 = K_t \cdot t\,; \quad K_3 = K_e \cdot w_c$$

This equation can be further simplified by omitting the two terms depending on C_2 and C_3. These two terms are expressions for the temperature change and dead load arch-shortening. For maximum stresses at the crown one must consider a drop in temperature which has the same sign and effect as the dead load arch-shortening. For simplification, assume both to be equal percentages of the factors, K_2 and K_3. All equations for the crown stresses, therefore, become:

$$\left.\begin{aligned} f_{c\,(\text{upper})} &= C_1 \frac{l^2}{d_c\, r} + 6\, K_1 \frac{l^2}{d_c^2} + 6\,(1 - \psi_c) \left(K_2 \frac{d_c}{r} + K_3 \frac{l^2}{r^2} \right) \\ f_{c\,(\text{lower})} &= C_1 \frac{l^2}{d_c\, r} - 6\, K_1 \frac{l^2}{d_c^2} - 6\,(1 + \psi_c) \left(K_2 \frac{d_c}{r} + K_3 \frac{l^2}{r^2} \right) \end{aligned}\right\} \ldots (68)$$

Thus, although both these terms were omitted, they have been re-introduced through the factor, ψ_c, which varies between the values, 0.05 and 0.20, depending on the variation of m and the ratio of crown thickness to the rise.

For $m = 0.3$:

$$\psi_c = 0.80 \frac{d_c}{r}$$

The ruling fiber stress used in determining the crown thickness is that of the upper fiber. The equation for this stress is rewritten in the following form:

$$f_c = f_c' + 6\,(1 - \psi_c) \left(K_2 \frac{d_c}{r} + K_3 \frac{l^2}{r^2} \right) \ldots\ldots\ldots\ldots (69)$$

in which, f_c' is the uniform compression caused by the dead load and live load thrusts. Substituting Equation (69) for f_c (upper) in Equation (68) and solving for d_c:

$$d_c = \frac{C_1\, l^2}{2\, r\, f_c'} \left[1 + \sqrt{1 + \frac{24\, f_c'\, K_1}{\left(\frac{C_1\, l}{r^2} \right)^2}} \right] \ldots\ldots\ldots\ldots\ldots (70)$$

This equation is useful only to check a dimension for the crown previously estimated or guessed. This must first be done in order to obtain the factor, g, on which depends the shape of the arch center line. After obtaining all C and K coefficients, the value of f_c' is obtained from Equation (69) by substi-

tuting for f_c the allowable fiber stress for which the arch is to be designed. If this guess was correct, Equation (70) will prove it. If not, there will result a new and better value for d_c, which may be checked by Equation (70).

If the crown thickness is unknown and the designer has no means of finding how close his guess may be, there is still another and better method of determining the crown thickness.

$w_c = w_0 + \gamma\, d_c$, in which w_0 is the weight per foot of arch rib superimposed above the crown and γ is the density of the arch material (150 lb. per cu. ft. for concrete):

$$\begin{aligned} C_1 &= C_a\,(w_0 + \gamma\, d_c) + C_i \cdot p \\ &= C_1' + C_1'' \cdot d_c \end{aligned}$$

Substitute this equation for C_1 in Equation (68). Then, the first term becomes:

$$C_1' \frac{l^2}{d_c\, r} + C_1'' \frac{l^2}{r}$$

in which,

$$C_1' = C_a \cdot w_0 + C_i \cdot p \quad \text{and} \quad C_1'' = C_a \cdot \gamma$$

K_3 will undergo a similar change and, as before:

$$f_{c\,(\text{upper})} = C_1' \frac{l^2}{d_c\, r} + C_1'' \frac{l^2}{r} + 6\, K_1 \frac{l^2}{d_c^2}$$
$$+ 6\,(1 - \psi_c)\left(K_2 \frac{d_c}{r} + K_3' \frac{l^2}{r^2} + K_3'' \frac{l^2}{r^2}\, d_c\right)$$
$$f_{c\,(\text{lower})} = C_1' \frac{l^2}{d_c\, r} + C_1'' \frac{l^2}{r} - 6\, K_1 \frac{l^2}{d_c^2}$$
$$- 6\,(1 + \psi_c)\left(K_2 \frac{d_c}{r} + K_3' \frac{l^2}{r^2} + K_3'' \frac{l^2}{r^2}\, d_c\right)$$

in which the various coefficients are those previously given and $K_3' = K_e \cdot w_0$ and $K_3'' = K_e \cdot \gamma$.

An example, using the author's arch, will illustrate the use of these two formulas:

$l = 100$ ft. $\qquad m = 0.339$
$r = 15$ ft. $\qquad g = 4.70.$

$C_d = 0.1080 + 0.0190 \times 4.70 - 0.0005 \times 4.70^2 = 0.1863$
$K_l = 0.00426 + 0.00037 \times 4.70 = 0.0060$
$C_l = 0.0579 + 0.0035 \times 4.70 = 0.0744$
$K_t = -1\,037.1 + 4.56 \times 4.70 = -1\,015.7$
$K_e = 0.0408 + 0.0046 \times 4.70 = 0.0624$
$\psi_c = \dfrac{0.80 \times 1.5}{15} = 0.08$
$C_1' = 0.1863 \times 280 + 0.0744 \times 200 = 67.04$
$C_1'' = 0.1863 \times 150 = 27.95$
$K_1 = 0.0060 \times 200 = 1.20$
$K_2 = -1\,015.7 \times (-35) = 35\,549.50$

$$K_3' = 0.0624 \times 280 = 17.47$$
$$K_3'' = 0.0624 \times 150 = 9.36$$

$$f_{c\,(\text{upper})} = \frac{67.04 \times 100^2}{d_c \times 15} + \frac{27.95 \times 100^2}{15} + \frac{6 \times 1.20 \times 100^2}{d_c^2}$$
$$+ 5.52 \left(\frac{35\,549.5 + d_c}{15} + \frac{17.47 \times 100^2}{15^2} + \frac{9.36 \times 100^2 \times d_c}{15^2} \right)$$

$$f_{c\,(\text{lower})} = \frac{67.04 \times 100^2}{d_c \times 15} + \frac{27.95 \times 100^2}{15} - \frac{6 \times 1.20 \times 100^2}{d_c^2}$$
$$- 6.48 \left(\frac{35\,549.5\, d_c}{15} + \frac{17.47 \times 100^2}{15^2} + \frac{9.36 \times 100^2 \times d_c}{15^2} \right)$$

$$f_{c\,(\text{upper})} = 22\,900 + 15\,890\, d_c + \frac{44\,700}{d_c} + \frac{72\,000}{d_c^2}$$

$$f_{c\,(\text{lower})} = 13\,580 - 18\,700\, d_c + \frac{44\,700}{d_c} - \frac{72\,000}{d_c^2}$$

All these quantities are in foot-units, therefore the stresses will be pounds per square foot, which may be divided by 144 to obtain pounds per square inch. By substituting various values of d_c in these equations, the values given in Table 27 will be obtained.

TABLE 27.—Stresses in Arch Ring for Various Crown Depths.

Crown depth, in feet.	f_c (upper), in pounds per square inch.	f_c (lower), in pounds per square inch.
0.5	2 830	—1 350
1.0	1 080	— 225
1.5	753	— 116
2.0	660	— 135
2.5	639	— 186
3.0	649	— 247

These results are plotted in Fig. 64 and indicate that for a crown thickness of 18 in. with a uniform live load of 200 lb. per sq. ft. and a fall of temperature of 35° Fahr., an upper fiber stress of 753 lb. per sq. in. and a tension in the lower fiber of 116 lb. per sq. in. may be expected. For these conditions of loading and temperature, one would expect to obtain a minimum compression in the upper fiber for a crown thickness of 30 in., while a minimum tension in the lower fiber would be obtained for a crown thickness of 20 in.

Similar curves could be drawn for the springing line section, but such additional work would entail too much unnecessary work. For average conditions, $m = 0.3$, by which the springing thickness can be computed after choosing the crown thickness. If, then, it is desired to adhere to the same maximum compressive stress chosen for the crown, the following formula may be used to check the thickness, d_s:

$$d_s = \frac{C_1\, l^2}{2 \cdot r \cdot f_c \cdot \cos \phi_s} \left[1 + \sqrt{1 - \frac{4 \cdot f_c \cdot A}{\left[\frac{C_1\, l}{r \cdot \cos \phi_s} \right]^2}} \right]$$

in which,

$$A = 6\,K_1 + 6\left(1 - \frac{\psi_s}{\cos\phi_3}\right)\left(K_2\frac{d_c^{\,3}}{l^2\,r} + K_3\frac{d_c^{\,2}}{r^2}\right)$$

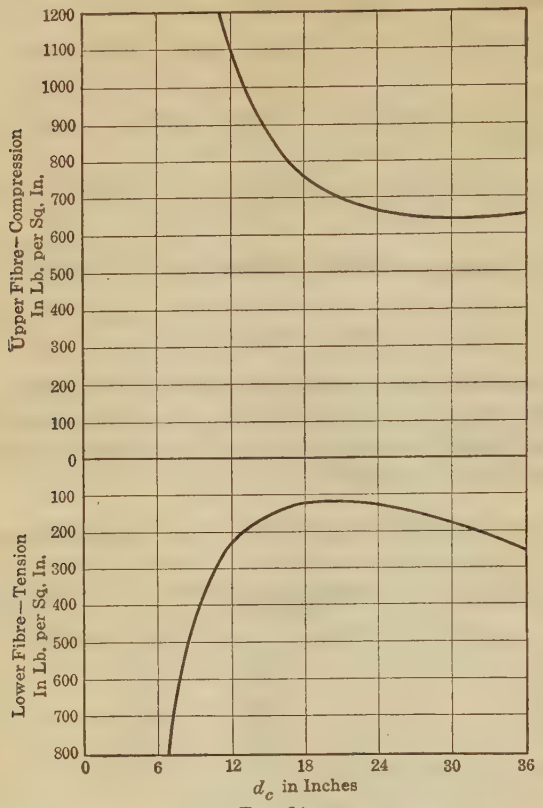

FIG. 64.

In these equations, the C and K coefficients must not be confused with those previously used for the crown thickness. For $m = 0.3$, these coefficients are:

C_d = same as for crown
$K_l = -0.02100 + (0.00069 \times g)$
$C_l = 0.0356 - (0.0008 \times g)$
$K_t = 3\,159.2 + (146.9 \times g)$
$K_e = -0.1129 - (0.0280 \times g)$
$\psi_s = 0.22\,\frac{d_s}{r}$
$C_1 = C_d \,.\, w_c + C_l \,.\, p$
$K_1 = K_l \,.\, p$
$K_2 = K_t \,.\, t°$
$K_3 = K_e \,.\, w_c$

If the rib is widened toward the springing line, the constants, C_1, K_1, K_2, and K_3, must be reduced by the ratio of the crown width to the width at the springing line.

The formulas here presented have been used by the writer for a number of years and have assisted him greatly in the design of reinforced concrete arch bridges (at least, as far as the paper design is concerned); but the serious student of such structures must often pause to wonder how close to reality his mathematical and computational efforts have come.

If the arch rib has been designed and executed as a line-of-thrust arch for the dead load, there remain the questions of arch-shortening and abutment displacements, shrinkage, and a possible internal re-organization of the arch due to "flow" or the "time-yield" of concrete, as it is most frequently called. There is a strong possibility that this latter property of concrete permits a readjustment of internal stresses and may even reduce temperature stresses, due to annual waves of temperature changes. At any rate, some mysterious forces are active in every arch, which cause readjustments of stresses, otherwise many arches now in use would collapse or, at best, be filled with cracks. The writer has had occasion to observe the crown deflections of several arches over a period of years and has come to the conclusion that if the stresses which mathematically correspond to these deflections were not relieved in some way, serious damage would be caused.

Then there is the question of temperature stresses. As previously mentioned, it is impossible to state definitely at what temperature there is no temperature thrust nor ensuing temperature moments. When the arch is concreted, sufficient heat is generated by the setting of the cement to raise the temperature of the mass of concrete. Temperatures as high as 128° Fahr. have been found in arches. If a large arch is poured in a few days in summer, what effect has such a rise of temperature when winter comes?

The writer also questions the uncertainty of the usual live load specifications for highway bridges. In the example, a live load of 200 lb. per sq. ft. was used for demonstration only. This load seems absurdly large for a 100 ft. span. Most proposed loadings would call for a uniform live load of about 100 lb. per sq. ft. for a span of that size. With four positions of the live load for maximum and minimum fiber stresses at any section, it stands to reason that a different loading is necessary for the hingeless arch than would apply to steel trusses.

With all these briefly mentioned uncertainties, the writer believes the high designing stresses in use to be deplorable. In the past ten years or more, the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete has reduced reinforced concrete column stresses, yet reinforced concrete arch rib stresses have been increased by individual engineers. This is due partly to the competition with steel bridges and partly to the engineer's desire to gain a reputation for designing cheap or economical bridges. The use of high unit stresses has resulted in the construction of stilt-like ribbed structures widely separated, with the roadway carried from rib to rib by long girders. At the crowns of the ribs, these long girders are usually rigidly connected to the arches, without any thought being given to the torsion developed in the ribs due to deflections in the floor-beams. Such structures may also be subjected to very high torsions due to wind pressures,

yet no thought is expended on even investigating such stresses. Several years ago, in France, a reinforced concrete arch, celebrated for its slenderness, was destroyed during a wind storm.

In conclusion, the writer does not wish to appear as an alarmist although there are many reasons for feeling uneasy. He merely desires to point out that there were other things to consider aside from the mathematics of the elastic theory, things which the novice has not had the time nor the experience to consider.

CHARLES W. COMSTOCK,* M. AM. SOC. C. E. (by letter).—The writer desires to record an emphatic protest against an expression occurring in the first sentence of this paper.

The term "elastic theory", which has unfortunately fallen into common use, is a careless and quite unnecessary short-cut for "mathematical theory of elasticity". It conveys a meaning very different than that intended, although often nearer the truth than the correct expression, the theory of elasticity having been frequently twisted and otherwise distorted beyond all recognition to meet some imaginary need for simplicity.

That the mathematical relationships on which engineers depend for calculated results are often complicated is no argument against their exactness or their use. Simplified formulas are much to be desired, but they can be justified only by detailed examination of each neglected or altered term and demonstration that its maximum value in any case is less than the allowable limit of error.

The theory of elasticity is by no means elastic. It is one of the most complete, consistent, and satisfactory in the whole range of mathematical physics within the limits set by the premises on which it is founded. These are simple, predicating only the proportionality of stress to strain and the property named elasticity, *viz.*, that the strained body will return to its original state upon removal of the stress. In many cases even the latter assumption is not a necessary prerequisite.

True, many problems lead to mathematical difficulties hitherto insuperable, and any presentation should be welcome that afforded solution of a problem which so far has proved baffling.

In examining the paper, the writer fails to find a new principle, a new result, the solution of a new problem, or even an improved solution of an old problem. By far the larger part of the text is devoted to the derivation of familiar formulas and the solution of problems. The remainder sets out a series of tables and diagrams the apparent intention of which is to standardize all arch proportioning in accordance with the author's views, leaving nothing to future designers but to take out the needed results as one would use a table of logarithms. The futility of such an effort should be evident. No two engineering problems are alike. Each presents its own peculiar features which must be studied as they appear and met with whatever resources and ingenuity the engineer can command. Cut and dried standards, even if they were

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feasible, dampen the engineer's enthusiasm, destroy his incentive, and kill any possibility of improvement in design or advancement in the art of construction.

The writer does not believe that the last word has been said on the calculation of stresses or the proportioning of structures. Until that time shall come, if ever, the greatest advancement will be made by encouraging each engineer to solve his problems in his own way with full knowledge of what others have done before. The duty of the Society and of its members is to keep the world informed of new problems, new solutions, and new discoveries.

In 1879 the scholarly Italian engineer Castigliano published his "Théorie de l'Equilibre des Systèmes Elastiques et ses Applications". No more concise, systematic, or exhaustive treatment of engineering uses of the theory of elasticity can be imagined. In the course of many years' study of such problems the writer has never encountered one that, if solvable at all, was not amenable to the principles and methods set forth in that masterly treatise, and without more labor than is required by the so-called modern methods.

In the nearly half century which has elapsed since that classic publication there has been a veritable flood of textbooks, treatises, and so-called scientific papers which have purported to present something new, but which for the most part have recorded merely the mental contortions of the respective authors in grasping principles already fully developed.

The writer thinks the title of this paper is too comprehensive. "Design" consists in the choice of suitable materials and their proper arrangement to accomplish a given purpose. Stress calculation is a necessary step in this procedure, usually the simplest and often relatively unimportant. The other and more difficult problems rarely admit of unique solutions or mathematical determination. Much is left to the discretion and judgment of the engineer based on the experience of himself and others. If it were otherwise, engineers could be rated on the single standard of mathematical accomplishment, and any mathematician would be an engineer.

A. G. Hayden,* M. Am. Soc. C. E. (by letter).—This discussion is pertinent to the paper only in a general way; and yet the writer feels that attention should be called to a particular line of experimental investigation that is much needed before full advantage can be taken of the great advances recently made in this field of engineering mathematics. What follows applies particularly to short-span arch bridges on the design of which heavy concentrated live loads have considerable influence.

It is generally recognized that certain practical problems await solution before accuracy of mathematical analysis becomes the governing factor in the design of such bridges. All methods of analysis presuppose ideal conditions of loading that are seldom realized in practice. Accuracy is possible for conditions of loading that approach uniformity of distribution, but in the case of heavy concentrations the designer is faced with the necessity of making certain out-and-out assumptions as to their lateral distribution that have more effect on the results than even a considerable approximation in the method of analysis. Concentrated loads are not applied on a line over

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the entire width of the arch ring, so that the correct load per unit width cannot be calculated; and, thus far, no tests have been made to determine how a concentrated load is distributed throughout the structure or what part of the load should be assumed to be carried by the critical element for purposes of analysis. In many cases a proper assumption is the controlling factor in the design. Carefully prepared influence lines for a unit load traveling over the span are not very significant without a solution of the problem of distribution. The writer has analyzed several arches for which uniform loadings could be assumed that would agree closely with the expected traffic; but unusual concentrations produced by heavy loads that were expected to pass over the bridge occasionally controlled the design even though higher unit stresses were permitted. Various assumptions as to the lateral distribution of these concentrations, even though apparently reasonable in themselves, made considerable difference in the results. Here is a problem that must be attacked, and it is a realization of this fact that leads many engineers to the opinion that much of the refinement of mathematical processes as applied to structural design is of little value. This is certainly a wrong point of view because the truth will only be reached by the elimination of uncertainties and the sooner this can be accomplished, either by mathematical or experimental means, the better.

The next step, therefore, should be to determine experimentally how a concentrated load is distributed throughout the arch ring from the spandrel columns of an open spandrel arch or through the earth-fill and over the arch ring of a filled spandrel arch. This is a baffling problem for the designer; a correct solution is necessary in order to give any considerable degree of consistency to the design of arch bridges.

A beginning has been made by a study of the laws of distribution of concentrated loads over flat slabs, a study that throws a little light on the problem as applied to arches as well. This refers to the so-called Illinois tests* by W. A. Slater, M. Am. Soc. C. E., and to the tests by the Ohio Highway Commission. These tests, however, are insufficient and inconclusive even as they relate to flat slabs in that (1) their range was entirely too limited, and (2) they failed to consider either non-central loading, or the shearing and diagonal tension stresses at the support, for loads near the support of slabs proportioned for moment in accordance with the determined law of distribution of the central load. The scope of these tests should be extended and then the investigator will be in better position to attack the problem as concerns the arch.

The author cannot receive too much praise for his paper since he has developed the first complete and satisfactory method for actually designing arch bridges as contrasted with methods of analyzing arches of assumed proportions; and there seem to be no loose ends in the development. There may be only a few engineers who have sufficient mathematical ability to follow through all his derivations (the writer does not pretend to be one of the few), but his results can be applied. The exposition and diagrams visualize

* Referred to in *Engineering News*, July 30, 1914, and discussed by Mr. Slater in the *Journal*, Am. Concrete Inst., January, 1914.

the behavior of arches under load in a manner that common methods of analysis do not accomplish. His investigation of the relative economy of arches having different ratios of crown thickness to thickness at the springing line is important.

H. E. WARRINGTON,* M. AM. SOC. C. E. (by letter).—With respect to this valuable paper and to the discussion by Professor Cain concerning the assumptions used in determining the increase in loading on the arch in terms of the crown loading, the writer has compared two open spandrel arch ribs with the formula given. These arches have spans of 94 and 100 ft. In the case of the 94-ft. span, it was impracticable to obtain the loading per foot at the springing line of the arch, and, therefore, that obtaining at the last column (a point one-tenth of the span from the springing line) was used, of course, with its corresponding y.

For the other span, of 100 ft., the loading was computed at the springing line in the manner outlined by the author.† The comparison has been made both for dead load only, and for dead load plus one-half the live load. The results are as shown in Table 28. As will be seen, the loads as computed by the formula differ from the actual loads by a maximum of about 16%, being larger in one case and smaller in the other.

TABLE 28.—RELATION OF ACTUAL AND COMPUTED ARCH LOADINGS.

100-FT. ARCH.

Point.	y.	DEAD LOAD.			DEAD LOAD PLUS ONE-HALF LIVE LOAD.		
		Actual, per linear foot.	By formula, per linear foot.	Percentage of actual.	Actual, per linear foot.	By formula, per linear foot.	Percentage of actual.
Crown.....	0.0	4 180	4 180	....	4 705	4 705	....
1	2.67	4 610	4 645	101	5 135	5 170	101
2	7.92	4 820	5 560	115	5 345	6 085	114
3	16.67	6 150	7 080	115	6 675	7 605	114
4 (Springing line)	30.00	9 400	9 400	....	9 935	9 935	....

94-FT. ARCH.

Point.	y.	DEAD LOAD.			DEAD LOAD PLUS ONE-HALF LIVE LOAD.		
		Actual, per linear foot.	By formula, per linear foot.	Percentage of actual.	Actual, per linear foot.	By formula, per linear foot.	Percentage of actual.
Crown.....	0.0	5 780	5 780	....	7 160	7 160	....
1	1.24	7 220	6 100	84.5	8 600	7 460	86.8
2	5.22	8 420	7 050	83.7	9 800	8 440	86.0
3	12.33	9 650	8 780	91.0	11 080	10 060	91.2
4	22.96	11 370	11 370	....	12 750	12 750	....
Springing line	37.50	.....	.....	....	.....	.....	....

* Cons. Engr. (Warrington & Kromer), San Francisco, Calif.

† See p. 970.

Both arch ribs have a large rise ratio, 30% of the span for the 100-ft. arch, and practically 40% for the 94-ft. arch. For the latter arch, the co-ordinates of the rib axis differ from those required by Table 6 by only about 0.1 ft. For the 100-ft. arch the agreement is not so close; the maximum difference, however, is only about 0.3 ft.

ROWLAND C. ROBIN,* JUN. AM. SOC. C. E. (by letter).—This paper fills a real need in presenting to engineers a means of designing symmetrical arches less complicated than the much used arch-summation method with its laborious trial-and-error procedure. Although this graphical method has a ready application, the writer feels that the analytical solution requires mathematics that is over the head of the average practical engineer. Further, the expressions for the curve of the arch axis and for the variation of the moment of inertia are a little complicated for every-day use.

A method of analytical design for symmetrical hingeless arches was developed by the writer and subsequently published by the Institution of Engineers, Australia.† Since this publication is not readily available to most American engineers, the writer ventures to describe the method here.

The type of arch used follows accepted practice and gives straightforward formulas which are readily proved as shown in the following brief outline.

The curve for the arch axis was derived by assuming the dead load to vary parabolically from the crown to the abutment (Fig. 65). If the dead load per linear foot at the crown of the rib is P lb. and that over the springing line is $P + Q$ lb., then that at a horizontal distance, x, from the crown will be, $P + \frac{Q}{l^2} x^2$, in which l is one-half the span. From this value of the dead load may readily be obtained by simple integration an equation for the arch axis of the type:

$$y = a x^2 + \frac{b}{l^2} x^4$$

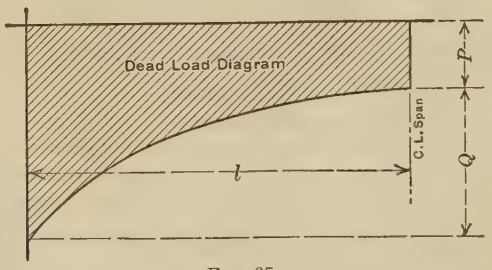

FIG. 65.

In this case, $\frac{b}{a} = \frac{Q}{6P}$, and an approximate value of the dead load thrust is $\frac{l^2}{r}\left(\frac{6P + Q}{12}\right)$, in which r is the rise of the axis.

* With Stone & Webster, Inc., Philadelphia, Pa.

† *Transactions*, The Institution of Engrs., Australia, Vol. II (1921).

The moments of inertia of the arch sections are made to vary according to the relation,

$$I = \frac{I_0 \sec \phi}{1 - \frac{1}{n}\frac{x^2}{l^2}}$$

in which,

x = horizontal distance of a point on the axis from the crown.
I = moment of inertia of the arch section at this point.
ϕ = angle of inclination of the arch axis with the horizontal.
I_0 = the moment of inertia of the crown-section.
$\frac{1}{n}$ = some number less than unity.

The three fundamental arch equations as usually stated are:

$$2\int_0^l M_0 \frac{d\,s}{I} + 2\int_0^l H_0 \frac{y\,d\,s}{I} = \int m \frac{d\,s}{I} \text{ (whole span)} \ldots\ldots (71)$$

$$2\int_0^l M_0\, y \frac{d\,s}{I} + 2\int_0^l H_0\, y^2 \frac{d\,s}{I} = \int m\, y \frac{d\,s}{I} \text{ (whole span)} \ldots (72)$$

$$2\int_0^l V_0 \frac{x^2\, d\,s}{I} = \int m\, x \frac{d\,s}{I} \text{ (whole span)} \ldots\ldots\ldots\ldots (73)$$

in which, M_0, H_0, V_0 are, respectively, the moment, horizontal thrust, and shear at the crown. The origin of co-ordinates is at the crown of the arch axis, and m is the moment about any point on the axis of all loads between it and the crown.

After integrating and solving the Equations (71), (72), and (73):

$$H_0 = \frac{A\,L_k - B\,C_k}{a\,S\,(A\,D - B^2)}$$

$$M_0 = \frac{l}{2\,A}\,(C_k - B_a\,S\,H_0)$$

$$= \frac{l}{2}\,\frac{D\,C_k - B\,L_k}{A\,D - B^2}$$

$$V_0 = \frac{G_k}{F}$$

in which,

S = span of arch axis.

$$A = \frac{1}{l}\left[I_0 \int_0^l \frac{d\,s}{I}\right] = 1 - \frac{1}{3\,n}$$

$$B = \frac{1}{a\,l^3}\left[I_0 \int_0^l y \frac{d\,s}{I}\right] = \left(\frac{1}{3} - \frac{1}{5\,n}\right) + \frac{b}{a}\left(\frac{1}{5} - \frac{1}{7\,n}\right)$$

$$D = \frac{1}{a^2\, l^5}\left[I_0 \int_0^l y^2\, \frac{d\,s}{I}\right]$$

$$= \left(\frac{1}{5} - \frac{1}{7\,n}\right) + \frac{b}{a}\left(\frac{2}{7} - \frac{2}{9\,n}\right) + \frac{b^2}{a^2}\left(\frac{1}{9} - \frac{1}{11\,n}\right)$$

$$F = \frac{2\,I_0}{l^3} \int_0^l x^2\, \frac{d\,s}{I} = 2\left(\frac{1}{3} - \frac{1}{5\,n}\right)$$

$$C_k = \frac{I_0}{l^2} \int m\, \frac{d\,s}{I} \text{ (whole span)}$$

$$G_k = \frac{I_0}{l^3} \int m\, x\, \frac{d\,s}{I} \text{ (whole span)}$$

$$L_k = \frac{I_0}{a\, l^4} \int m\, y\, \frac{d\,s}{I} \text{ (whole span)}$$

For a unit load at a horizontal distance, $k\,l$, from the crown, the variable factors, C_k, G_k, and L_k, are as follows:

$$C_k = \left(\frac{1-k}{2}\right)^2 - \frac{1}{n}\left(\frac{3 - 4\,k + k^4}{12}\right)$$

$$G_k = \frac{3 - 4\,k + k^4}{6} - \frac{1}{n}\left(\frac{4 - 5\,k + k^5}{20}\right)$$

$$L_k = \frac{3 - 4\,k + k^4}{12} - \frac{1}{n}\left(\frac{5 - 6\,k + k^6}{30}\right)$$

$$+ \frac{b}{a}\left(\frac{5 - 6\,k + k^6}{30}\right) - \frac{b}{a} \cdot \frac{1}{n}\left(\frac{7 - 8\,k + k^8}{56}\right)$$

If the load is of a unit intensity, distributed over one-half the span:

$$C_k = \left(\frac{1}{6} - \frac{1}{10\,n}\right)$$

$$G_k = \left(\frac{1}{8} - \frac{1}{12\,n}\right)$$

$$L_k = \left(\frac{1}{10} - \frac{1}{14\,n} + \frac{b}{a}\left(\frac{1}{14} - \frac{1}{18\,n}\right)\right.$$

If a similar load is distributed over one-third of the "half span" from the crown:

$$C_k = \left(\frac{19}{54} - \frac{1}{n} \cdot \frac{473}{2\ 430}\right)$$

$$G_k = \left(\frac{163}{648} - \frac{1}{n} \cdot \frac{2\ 309}{14\ 580}\right)$$

$$L_k = \left(\frac{473}{2\ 430} - \frac{1}{n}\,\frac{20\ 413}{153\ 090}\right) + \frac{b}{a}\left(\frac{20\ 413}{153\ 090} - \frac{1}{n} \cdot \frac{83\ 653}{826\ 686}\right)$$

Also, for temperature changes:

$$H_0 = \pm \frac{I_0 E A T \alpha}{a^2 l^4 (A D - B^2)}$$

and,

$$M_0 = - H_0 \frac{B a l^2}{A}$$

in which,

α = the coefficient of expansion of the material of the arch; and
T = the rise or fall of temperature, in degrees Fahrenheit.

These results, while not simple, are, nevertheless, not too unwieldy for use and, further, lend themselves readily to tabulation or plotting.

The form of the arch rib is slightly more slender than that presented by the author, using approximately equal thicknesses at the crown and the springing line.

For temperature effects using the tables included in the publication previously mentioned, the writer found that:

$$H_0 = 53\ T$$

$$M_0 > - 169\ T$$

for an arch of a span of 100 ft. and a rise of 15 ft., using an I_0 of 6 399 in. units and values of $\frac{b}{a}$ and $\frac{1}{n}$ equal to 0.5 and 0.7, respectively. These results compare with:

$$M_0 = - 185.0\ t$$

$$H_0 = 58.9\ t$$

found graphically by the author and show a small but appreciable saving.

A curve of the type,

$$y = a x^2 + \frac{b}{l^2} x^4$$

for a span of 100 ft. and rise of 15 ft., shows a maximum divergence from the author's curve, of $1\frac{3}{8}$ in., the value of $\frac{b}{a}$ having been calculated (from the loads given) to be approximately 0.5.

In actual design according to this method, Tables 29 to 35, inclusive,* will be found useful. For any given arch, the factors, A, B, D, and F, are constant, but the factors C_k, G_k, and L_k depend on the conditions of loading. The tables give sufficient load positions to include all cases.

* From *Transactions*, The Institution of Engrs., Australia, Vol. II (1921).

TABLE 29.—VALUES OF VARIABLE, $C_k \times 100\,000$, FOR UNIT LOAD AT DISTANCE, $k\,l$, FROM CROWN, FOR DIFFERENT VALUES OF $\frac{1}{n}$, AND FOR HALF-ARCH SUBDIVISIONS FROM 20 TO 6.

20 SUBDIVISIONS OF HALF ARCH.

k.	Values of $\frac{1}{n}$.					
	0	0.5	0.6	0.7	0.8	0.9
0	50 000	37 500	35 000	32 500	30 000	27 500
0.05	45 125	33 458	31 125	28 792	26 458	24 125
0.10	40 500	29 666	27 500	25 333	23 167	20 999
0.15	36 125	26 129	24 122	22 122	20 122	18 121
0.20	32 000	22 827	20 992	19 157	17 323	15 488
0.25	28 125	19 775	18 105	16 435	14 766	13 096
0.30	24 500	16 966	15 460	13 953	12 446	10 939
0.35	21 125	14 396	13 050	11 704	10 358	9 012
0.40	18 000	12 060	10 872	9 684	8 496	7 308
0.45	15 125	9 954	8 920	7 886	6 852	5 818
0.50	12 500	8 073	7 187	6 302	5 417	4 531
0.55	10 125	6 410	5 667	4 924	4 182	3 439
0.60	8 000	4 960	4 352	3 741	3 136	2 528
0.65	6 125	3 715	3 232	2 750	2 268	1 786
0.70	4 500	2 666	2 300	1 943	1 566	1 199
0.75	3 125	1 807	1 543	1 279	1 016	752
0.80	2 000	1 127	958	777	603	428
0.85	1 125	617	515	413	312	210
0.90	500	266	220	173	126	79
0.95	125	65	52	40	28	16
1.00	0	0	0	0	0	0
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

11 SUBDIVISIONS OF HALF-ARCH.

k.	Values of $\frac{1}{n}$.					
	0	0.5	0.6	0.7	0.8	0.9
1/11	41 322	30 337	28 140	25 943	23 746	21 549
2/11	33 471	23 997	22 102	20 207	18 312	16 417
3/11	26 446	18 469	16 873	15 278	13 682	12 087
4/11	20 248	13 736	12 433	11 131	9 828	8 526
5/11	14 876	9 774	8 753	7 733	6 713	5 692
6/11	10 331	6 553	5 797	5 042	4 286	3 530
7/11	6 613	4 035	3 520	3 004	2 489	1 974
8/11	3 719	2 175	1 866	1 557	1 248	939
9/11	1 653	922	776	630	484	338
10/11	413	219	180	141	102	63

7 SUBDIVISIONS OF HALF-ARCH.

k.	0	0.5	0.6	0.7	0.8	0.9
1/7	36 735	26 614	24 590	22 566	20 541	18 517
2/7	25 510	17 744	16 191	14 638	13 085	11 532
3/7	16 327	10 829	9 729	8 630	7 530	6 431
4/7	9 184	5 763	5 079	4 395	3 711	3 087
5/7	4 082	2 402	2 066	1 730	1 394	1 058
6/7	1 020	557	464	372	279	186

9 SUBDIVISIONS OF HALF-ARCH.

k.	Values of $\frac{1}{n}$.					
	0	0.5	0.6	0.7	0.8	0.9
1/9	39 506	28 857	26 728	24 598	22 468	20 338
2/9	30 247	21 440	19 679	17 918	16 157	14 395
3/9	22 222	15 226	13 827	12 428	11 029	9 630
4/9	15 432	10 177	9 126	8 075	7 024	5 973
5/9	9 877	6 239	5 511	4 784	4 056	3 329
6/9	5 556	3 344	2 901	2 459	2 016	1 574
7/9	2 469	1 407	1 195	983	770	558
8/9	617	331	274	216	159	102
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

6 SUBDIVISIONS OF HALF-ARCH.

k.	0	0.5	0.6	0.7	0.8	0.9
1/6	34 722	24 997	23 052	21 107	19 161	17 216
1/8	22 222	15 226	13 827	12 428	11 029	9 630
1/2	12 500	8 073	7 187	6 302	5 417	4 581
2/6	5 556	3 344	2 901	2 459	2 016	1 574
5/6	1 389	768	644	520	396	272
....	.....	.....	.....	.....	.....	.....

8 SUBDIVISIONS OF HALF-ARCH.

k.	Values of $\frac{1}{n}$.					
	0	0.5	0.6	0.7	0.8	0.9
1/8	38 281	27 863	25 780	23 696	21 613	19 529
1/4	28 125	19 775	18 105	16 435	14 766	13 096
3/8	19 531	13 199	11 932	10 666	9 399	8 133
1/2	12 500	8 073	7 187	6 302	5 417	4 531
5/8	7 031	4 812	3 768	3 224	2 681	2 137
3/4	3 125	1 807	1 543	1 279	1 016	752
7/8	781	422	350	278	207	135
....	.....	.....	.....	.....	.....	.....
....	.....	.....	.....	.....	.....	.....

$$C_k = \frac{I_0}{l^2} \int m \frac{d\,s}{I} \text{ (whole span.)}$$

TABLE 30.—VALUES OF VARIABLE, $L_k \times 100\,000$, FOR UNIT LOAD AT DISTANCE, $k\,l$, FROM CROWN, FOR DIFFERENT VALUES OF $\frac{1}{n}$, AND FOR HALF-ARCH SUBDIVISIONS FROM 20 TO 6.

$$\left(L_k = \frac{I}{a\,l^4} \int m\,y\,\frac{d\,s}{I} \text{ (whole span)}\right)$$

20 SUBDIVISIONS OF HALF-ARCH.

k.	Values of $\frac{1}{n}$.					
	0	0.5	0.6	0.7	0.8	0.9
0	$25\,000 + 16\,667\,\frac{b}{a}$	$16\,667 + 10\,417\,\frac{b}{a}$	$15\,000 + 9\,167\,\frac{b}{a}$	$13\,333 + 7\,917\,\frac{b}{a}$	$11\,667 + 6\,667\,\frac{b}{a}$	$10\,000 + 5\,417\,\frac{b}{a}$
0.05	$23\,333 + 15\,667\,\frac{b}{a}$	$15\,500 + 9\,774\,\frac{b}{a}$	$13\,933 + 8\,595\,\frac{b}{a}$	$12\,367 + 7\,417\,\frac{b}{a}$	$10\,800 + 6\,238\,\frac{b}{a}$	$9\,233 + 5\,060\,\frac{b}{a}$
0.10	$21\,667 + 14\,667\,\frac{b}{a}$	$14\,334 + 9\,131\,\frac{b}{a}$	$12\,867 + 8\,024\,\frac{b}{a}$	$11\,400 + 6\,917\,\frac{b}{a}$	$9\,934 + 5\,810\,\frac{b}{a}$	$8\,467 + 4\,703\,\frac{b}{a}$
0.15	$20\,004 + 13\,667\,\frac{b}{a}$	$13\,171 + 8\,488\,\frac{b}{a}$	$11\,804 + 7\,452\,\frac{b}{a}$	$10\,437 + 6\,417\,\frac{b}{a}$	$9\,071 + 5\,381\,\frac{b}{a}$	$7\,704 + 4\,345\,\frac{b}{a}$
0.20	$18\,347 + 12\,667\,\frac{b}{a}$	$12\,013 + 7\,845\,\frac{b}{a}$	$10\,747 + 6\,881\,\frac{b}{a}$	$9\,480 + 5\,917\,\frac{b}{a}$	$8\,213 + 4\,953\,\frac{b}{a}$	$6\,946 + 3\,988\,\frac{b}{a}$
0.25	$16\,699 + 11\,667\,\frac{b}{a}$	$10\,805 + 7\,203\,\frac{b}{a}$	$9\,699 + 6\,310\,\frac{b}{a}$	$8\,532 + 5\,418\,\frac{b}{a}$	$7\,365 + 4\,525\,\frac{b}{a}$	$6\,198 + 3\,632\,\frac{b}{a}$
0.30	$15\,067 + 10\,669\,\frac{b}{a}$	$9\,733 + 6\,562\,\frac{b}{a}$	$8\,666 + 5\,740\,\frac{b}{a}$	$7\,599 + 4\,919\,\frac{b}{a}$	$6\,532 + 4\,098\,\frac{b}{a}$	$5\,465 + 3\,276\,\frac{b}{a}$
0.35	$13\,458 + 9\,673\,\frac{b}{a}$	$8\,622 + 5\,923\,\frac{b}{a}$	$7\,655 + 5\,173\,\frac{b}{a}$	$6\,687 + 4\,423\,\frac{b}{a}$	$5\,720 + 3\,673\,\frac{b}{a}$	$4\,753 + 2\,922\,\frac{b}{a}$
0.40	$11\,880 + 8\,680\,\frac{b}{a}$	$7\,540 + 5\,287\,\frac{b}{a}$	$6\,672 + 4\,608\,\frac{b}{a}$	$5\,804 + 3\,929\,\frac{b}{a}$	$4\,936 + 3\,251\,\frac{b}{a}$	$4\,068 + 2\,572\,\frac{b}{a}$
0.45	$10\,342 + 7\,694\,\frac{b}{a}$	$6\,495 + 4\,657\,\frac{b}{a}$	$5\,725 + 4\,050\,\frac{b}{a}$	$4\,956 + 3\,442\,\frac{b}{a}$	$4\,186 + 2\,835\,\frac{b}{a}$	$3\,417 + 2\,227\,\frac{b}{a}$
0.50	$8\,854 + 6\,719\,\frac{b}{a}$	$5\,495 + 4\,037\,\frac{b}{a}$	$4\,823 + 3\,500\,\frac{b}{a}$	$4\,151 + 2\,964\,\frac{b}{a}$	$3\,479 + 2\,427\,\frac{b}{a}$	$2\,807 + 1\,891\,\frac{b}{a}$
0.55	$7\,429 + 5\,769\,\frac{b}{a}$	$4\,550 + 3\,430\,\frac{b}{a}$	$3\,974 + 2\,964\,\frac{b}{a}$	$3\,398 + 2\,498\,\frac{b}{a}$	$2\,822 + 2\,033\,\frac{b}{a}$	$2\,246 + 1\,567\,\frac{b}{a}$
0.60	$6\,080 + 4\,822\,\frac{b}{a}$	$3\,669 + 2\,843\,\frac{b}{a}$	$3\,187 + 2\,447\,\frac{b}{a}$	$2\,704 + 2\,051\,\frac{b}{a}$	$2\,222 + 1\,655\,\frac{b}{a}$	$1\,740 + 1\,259\,\frac{b}{a}$
0.65	$4\,821 + 3\,918\,\frac{b}{a}$	$2\,862 + 2\,282\,\frac{b}{a}$	$2\,470 + 1\,955\,\frac{b}{a}$	$2\,078 + 1\,628\,\frac{b}{a}$	$1\,686 + 1\,301\,\frac{b}{a}$	$1\,295 + 974\,\frac{b}{a}$
0.70	$3\,667 + 3\,059\,\frac{b}{a}$	$2\,138 + 1\,757\,\frac{b}{a}$	$1\,832 + 1\,498\,\frac{b}{a}$	$1\,526 + 1\,237\,\frac{b}{a}$	$1\,220 + 976\,\frac{b}{a}$	$915 + 716\,\frac{b}{a}$
0.75	$2\,637 + 2\,260\,\frac{b}{a}$	$1\,507 + 1\,278\,\frac{b}{a}$	$1\,281 + 1\,081\,\frac{b}{a}$	$1\,055 + 885\,\frac{b}{a}$	$829 + 688\,\frac{b}{a}$	$603 + 492\,\frac{b}{a}$
0.80	$1\,747 + 1\,540\,\frac{b}{a}$	$976 + 855\,\frac{b}{a}$	$822 + 718\,\frac{b}{a}$	$668 + 581\,\frac{b}{a}$	$514 + 444\,\frac{b}{a}$	$360 + 307\,\frac{b}{a}$
0.85	$1\,017 + 924\,\frac{b}{a}$	$555 + 502\,\frac{b}{a}$	$462 + 418\,\frac{b}{a}$	$370 + 333\,\frac{b}{a}$	$278 + 249\,\frac{b}{a}$	$185 + 164\,\frac{b}{a}$
0.90	$467 + 438\,\frac{b}{a}$	$248 + 232\,\frac{b}{a}$	$205 + 191\,\frac{b}{a}$	$161 + 150\,\frac{b}{a}$	$117 + 109\,\frac{b}{a}$	$72 + 68\,\frac{b}{a}$
0.95	$121 + 117\,\frac{b}{a}$	$62 + 60\,\frac{b}{a}$	$51 + 49\,\frac{b}{a}$	$39 + 38\,\frac{b}{a}$	$27 + 26\,\frac{b}{a}$	$16 + 15\,\frac{b}{a}$
1.00	0	0	0	0	0	0

TABLE 30.—(*Continued.*)

11 Subdivisions of Half-Arch.

k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
1/11	$21\,970 + 14\,848\frac{b}{a}$	$14\,546 + 9\,248\frac{b}{a}$	$13\,061 + 8\,128\frac{b}{a}$	$11\,576 + 7\,008\frac{b}{a}$	$10\,092 + 5\,887\frac{b}{a}$	$8\,607 + 4\,767\frac{b}{a}$
2/11	$18\,948 + 13\,030\frac{b}{a}$	$12\,433 + 8\,079\frac{b}{a}$	$11\,130 + 7\,089\frac{b}{a}$	$9\,827 + 6\,099\frac{b}{a}$	$8\,524 + 5\,108\frac{b}{a}$	$7\,221 + 4\,118\frac{b}{a}$
3/11	$15\,955 + 11\,213\frac{b}{a}$	$10\,348 + 6\,911\frac{b}{a}$	$9\,227 + 6\,051\frac{b}{a}$	$8\,106 + 5\,191\frac{b}{a}$	$6\,948 + 4\,330\frac{b}{a}$	$5\,863 + 3\,470\frac{b}{a}$
4/11	$13\,024 + 9\,402\frac{b}{a}$	$8\,324 + 5\,749\frac{b}{a}$	$7\,383 + 5\,018\frac{b}{a}$	$6\,443 + 4\,288\frac{b}{a}$	$5\,503 + 3\,557\frac{b}{a}$	$4\,563 + 2\,826\frac{b}{a}$
5/11	$10\,204 + 7\,605\frac{b}{a}$	$6\,402 + 4\,600\frac{b}{a}$	$5\,641 + 3\,999\frac{b}{a}$	$4\,880 + 3\,398\frac{b}{a}$	$4\,120 + 2\,797\frac{b}{a}$	$3\,359 + 2\,196\frac{b}{a}$
6/11	$7\,556 + 5\,845\frac{b}{a}$	$4\,683 + 3\,484\frac{b}{a}$	$4\,049 + 3\,012\frac{b}{a}$	$3\,464 + 2\,540\frac{b}{a}$	$2\,879 + 2\,068\frac{b}{a}$	$2\,295 + 1\,596\frac{b}{a}$
7/11	$5\,154 + 4\,161\frac{b}{a}$	$3\,074 + 2\,432\frac{b}{a}$	$2\,658 + 2\,086\frac{b}{a}$	$2\,242 + 1\,741\frac{b}{a}$	$1\,826 + 1\,395\frac{b}{a}$	$1\,410 + 1\,049\frac{b}{a}$
8/11	$3\,089 + 2\,614\frac{b}{a}$	$1\,782 + 1\,489\frac{b}{a}$	$1\,520 + 1\,264\frac{b}{a}$	$1\,259 + 1\,039\frac{b}{a}$	$997 + 814\frac{b}{a}$	$736 + 589\frac{b}{a}$
9/11	$1\,462 + 1\,303\frac{b}{a}$	$810 + 718\frac{b}{a}$	$680 + 601\frac{b}{a}$	$550 + 484\frac{b}{a}$	$419 + 367\frac{b}{a}$	$289 + 250\frac{b}{a}$
10/11	$389 + 366\frac{b}{a}$	$206 + 193\frac{b}{a}$	$169 + 159\frac{b}{a}$	$132 + 124\frac{b}{a}$	$96 + 90\frac{b}{a}$	$59 + 55\frac{b}{a}$

9 Subdivisions of Half-Arch.

1/9	$21\,298 + 14\,444\frac{b}{a}$	$14\,075 + 8\,988\frac{b}{a}$	$12\,631 + 7\,897\frac{b}{a}$	$11\,187 + 6\,806\frac{b}{a}$	$9\,742 + 5\,714\frac{b}{a}$	$8\,298 + 4\,623\frac{b}{a}$
2/9	$17\,613 + 12\,223\frac{b}{a}$	$11\,502 + 7\,560\frac{b}{a}$	$10\,279 + 6\,627\frac{b}{a}$	$9\,057 + 5\,695\frac{b}{a}$	$7\,835 + 4\,762\frac{b}{a}$	$6\,613 + 3\,830\frac{b}{a}$
3/9	$13\,992 + 10\,005\frac{b}{a}$	$8\,989 + 6\,135\frac{b}{a}$	$7\,989 + 5\,362\frac{b}{a}$	$6\,989 + 4\,588\frac{b}{a}$	$5\,988 + 3\,814\frac{b}{a}$	$4\,988 + 3\,040\frac{b}{a}$
4/9	$10\,510 + 7\,803\frac{b}{a}$	$6\,609 + 4\,727\frac{b}{a}$	$5\,828 + 4\,111\frac{b}{a}$	$5\,048 + 3\,496\frac{b}{a}$	$4\,268 + 2\,881\frac{b}{a}$	$3\,487 + 2\,265\frac{b}{a}$
5/9	$7\,275 + 5\,654\frac{b}{a}$	$4\,449 + 3\,364\frac{b}{a}$	$3\,883 + 2\,906\frac{b}{a}$	$3\,318 + 2\,448\frac{b}{a}$	$2\,752 + 1\,990\frac{b}{a}$	$2\,187 + 1\,532\frac{b}{a}$
6/9	$4\,424 + 3\,626\frac{b}{a}$	$2\,611 + 2\,103\frac{b}{a}$	$2\,248 + 1\,798\frac{b}{a}$	$1\,886 + 1\,494\frac{b}{a}$	$1\,523 + 1\,189\frac{b}{a}$	$1\,160 + 885\frac{b}{a}$
7/9	$2\,124 + 1\,849\frac{b}{a}$	$1\,199 + 1\,035\frac{b}{a}$	$1\,014 + 872\frac{b}{a}$	$829 + 709\frac{b}{a}$	$644 + 547\frac{b}{a}$	$460 + 384\frac{b}{a}$
8/9	$573 + 533\frac{b}{a}$	$306 + 284\frac{b}{a}$	$253 + 234\frac{b}{a}$	$200 + 185\frac{b}{a}$	$146 + 135\frac{b}{a}$	$93 + 85\frac{b}{a}$

TABLE 30.—(*Continued.*)

8 Subdivisions of Half-Arch.

k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
1/8	$20\,835 + 14\,167\frac{b}{a}$	$13\,752 + 8\,809\frac{b}{a}$	$12\,335 + 7\,738\frac{b}{a}$	$10\,919 + 6\,667\frac{b}{a}$	$9\,502 + 5\,596\frac{b}{a}$	$8\,085 + 4\,524\frac{b}{a}$
1/4	$16\,699 + 11\,667\frac{b}{a}$	$10\,865 + 7\,203\frac{b}{a}$	$9\,699 + 6\,310\frac{b}{a}$	$8\,532 + 5\,418\frac{b}{a}$	$7\,365 + 4\,525\frac{b}{a}$	$6\,198 + 3\,632\frac{b}{a}$
3/8	$12\,665 + 9\,176\frac{b}{a}$	$8\,077 + 5\,604\frac{b}{a}$	$7\,159 + 4\,890\frac{b}{a}$	$6\,242 + 4\,175\frac{b}{a}$	$5\,324 + 3\,461\frac{b}{a}$	$4\,406 + 2\,747\frac{b}{a}$
1/2	$8\,854 + 6\,719\frac{b}{a}$	$5\,495 + 4\,037\frac{b}{a}$	$4\,823 + 3\,500\frac{b}{a}$	$4\,151 + 2\,964\frac{b}{a}$	$3\,479 + 2\,427\frac{b}{a}$	$2\,807 + 1\,891\frac{b}{a}$
5/8	$5\,438 + 4\,365\frac{b}{a}$	$3\,256 + 2\,559\frac{b}{a}$	$2\,819 + 2\,197\frac{b}{a}$	$2\,382 + 1\,836\frac{b}{a}$	$1\,946 + 1\,475\frac{b}{a}$	$1\,509 + 1\,114\frac{b}{a}$
3/4	$2\,637 + 2\,260\frac{b}{a}$	$1\,507 + 1\,278\frac{b}{a}$	$1281 + 1\,081\frac{b}{a}$	$1\,055 + 885\frac{b}{a}$	$829 + 688\frac{b}{a}$	$603 + 492\frac{b}{a}$
7/8	$718 + 663\frac{b}{a}$	$386 + 356\frac{b}{a}$	$321 + 295\frac{b}{a}$	$254 + 233\frac{b}{a}$	$188 + 172\frac{b}{a}$	$122 + 111\frac{b}{a}$

7 Subdivisions of Half-Arch.

1/7	$20\,242 + 13\,809\frac{b}{a}$	$13\,337 + 8\,580\frac{b}{a}$	$11\,956 + 7\,534\frac{b}{a}$	$10\,575 + 6\,488\frac{b}{a}$	$9\,194 + 5\,442\frac{b}{a}$	$7\,813 + 4\,396\frac{b}{a}$
2/7	$15\,532 + 10\,954\frac{b}{a}$	$10\,055 + 6\,745\frac{b}{a}$	$8\,959 + 5\,903\frac{b}{a}$	$7\,864 + 5\,061\frac{b}{a}$	$6\,768 + 4\,219\frac{b}{a}$	$5\,673 + 3\,378\frac{b}{a}$
3/7	$10\,995 + 8\,116\frac{b}{a}$	$6\,937 + 4\,926\frac{b}{a}$	$6\,126 + 4\,288\frac{b}{a}$	$5\,314 + 3\,650\frac{b}{a}$	$4\,502 + 3\,012\frac{b}{a}$	$3\,691 + 2\,374\frac{b}{a}$
4/7	$6\,841 + 5\,354\frac{b}{a}$	$4\,164 + 3\,176\frac{b}{a}$	$3\,628 + 2\,740\frac{b}{a}$	$3\,093 + 2\,304\frac{b}{a}$	$2\,558 + 1\,869\frac{b}{a}$	$2\,022 + 1\,433\frac{b}{a}$
5/7	$3\,360 + 2\,824\frac{b}{a}$	$1\,948 + 1\,615\frac{b}{a}$	$1\,665 + 1\,373\frac{b}{a}$	$1\,383 + 1\,132\frac{b}{a}$	$1\,101 + 890\frac{b}{a}$	$818 + 648\frac{b}{a}$
6/7	$927 + 846\frac{b}{a}$	$504 + 458\frac{b}{a}$	$419 + 380\frac{b}{a}$	$335 + 303\frac{b}{a}$	$250 + 225\frac{b}{a}$	$166 + 148\frac{b}{a}$

6 Subdivisions of Half-Arch.

1/6	$19\,451 + 13\,333\frac{b}{a}$	$12\,784 + 8\,274\frac{b}{a}$	$11\,451 + 7\,262\frac{b}{a}$	$10\,118 + 6\,250\frac{b}{a}$	$8\,784 + 5\,238\frac{b}{a}$	$7\,451 + 4\,226\frac{b}{a}$
1/3	$13\,992 + 10\,005\frac{b}{a}$	$8\,989 + 6\,135\frac{b}{a}$	$7\,989 + 5\,362\frac{b}{a}$	$6\,989 + 4\,588\frac{b}{a}$	$5\,988 + 3\,814\frac{b}{a}$	$4\,988 + 3\,040\frac{b}{a}$
1/2	$8\,851 + 6\,719\frac{b}{a}$	$5\,495 + 4\,037\frac{b}{a}$	$4\,823 + 3\,500\frac{b}{a}$	$4\,151 + 2\,964\frac{b}{a}$	$3\,479 + 2\,427\frac{b}{a}$	$2\,807 + 1\,891\frac{b}{a}$
2/3	$4\,424 + 3\,626\frac{b}{a}$	$2\,611 + 2\,103\frac{b}{a}$	$2\,248 + 1\,798\frac{b}{a}$	$1\,886 + 1\,494\frac{b}{a}$	$1\,523 + 1\,189\frac{b}{a}$	$1\,161 + 885\frac{b}{a}$
5/6	$1\,241 + 1\,116\frac{b}{a}$	$683 + 611\frac{b}{a}$	$571 + 510\frac{b}{a}$	$460 + 409\frac{b}{a}$	$348 + 308\frac{b}{a}$	$236 + 207\frac{b}{a}$

TABLE 31.—VALUES OF VARIABLE, $G_k \times 100\,000$, FOR UNIT LOAD AT DISTANCE, $k\,l$, FROM CROWN, FOR DIFFERENT VALUES OF $\frac{1}{n}$, AND FOR HALF-ARCH SUBDIVISIONS FROM 20 TO 6.

20 Subdivisions of Half Arch.						
k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
0.00	33 333	23 333	21 333	19 333	17 333	15 333
0.05	30 835	21 460	19 585	17 710	15 835	13 960
0.10	28 350	19 600	17 850	16 100	14 350	12 600
0.15	25 890	17 764	16 139	14 514	12 889	11 264
0.20	23 467	15 966	14 466	12 966	11 465	9 965
0.25	21 094	14 216	12 841	11 465	10 090	8 714
0.30	18 783	12 527	11 276	10 025	8 773	7 522
0.35	16 548	10 910	9 782	8 654	7 527	6 399
0.40	14 400	9 374	8 369	7 364	6 359	5 354
0.45	12 352	7 931	7 047	6 162	5 278	4 394
0.50	10 417	6 589	5 823	5 051	4 292	3 526
0.55	8 606	5 355	4 705	4 055	3 405	2 755
0.60	6 933	4 239	3 700	3 161	2 622	2 088
0.65	5 410	3 245	2 812	2 379	1 946	1 513
0.70	4 050	2 380	2 046	1 712	1 378	1 044
0.75	2 865	1 646	1 403	1 159	915	672
0.80	1 867	1 048	884	720	556	392
0.85	1 069	584	488	391	294	197
0.90	483	257	212	167	121	76
0.95	123	64	52	40	28	16
1.00	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

11 Subdivisions of Half Arch.						
k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
$\frac{1}{11}$	28 800	19 937	18 164	16 391	14 619	12 846
$\frac{2}{11}$	24 343	16 615	15 069	13 524	11 978	10 433
$\frac{3}{11}$	20 035	13 440	12 121	10 803	9 484	8 165
$\frac{4}{11}$	15 953	10 482	9 388	8 294	7 200	6 106
$\frac{5}{11}$	12 171	7 805	6 931	6 058	5 185	4 311
$\frac{6}{11}$	8 765	5 463	4 802	4 142	3 481	2 821
$\frac{7}{11}$	5 810	3 504	3 043	2 581	2 120	1 659
$\frac{8}{11}$	3 381	1 963	1 680	1 396	1 112	829
$\frac{9}{11}$	1 553	863	725	588	450	312
$\frac{10}{11}$	401	212	174	137	99	61

7 Subdivisions of Half Arch.						
k.	0	0.5	0.6	0.7	0.8	0.9
$\frac{1}{7}$	26 239	18 025	16 382	14 739	13 096	11 453
$\frac{2}{7}$	19 436	13 003	11 716	10 430	9 143	7 856
$\frac{3}{7}$	13 217	8 538	7 602	6 666	5 730	4 794
$\frac{4}{7}$	7 872	4 862	4 260	3 658	3 057	2 455
$\frac{5}{7}$	3 693	2 157	1 849	1 542	1 235	928
$\frac{6}{7}$	972	529	441	352	264	176

9 Subdivisions of Half Arch.						
k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
$\frac{1}{9}$	27 801	19 189	17 467	15 745	14 023	12 301
$\frac{2}{9}$	22 405	15 182	13 737	12 292	10 847	9 403
$\frac{3}{9}$	17 284	11 440	10 272	9 103	7 934	6 765
$\frac{4}{9}$	12 574	8 087	7 189	6 291	5 394	4 496
$\frac{5}{9}$	8 413	5 223	4 588	3 950	3 312	2 675
$\frac{6}{9}$	4 938	2 942	2 543	2 144	1 745	1 346
$\frac{7}{9}$	2 286	1 297	1 099	901	708	505
$\frac{8}{9}$	594	318	263	208	152	97
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

6 Subdivisions of Half Arch.						
k.	0	0.5	0.6	0.7	0.8	0.9
$\frac{1}{6}$	25 077	17 160	15 577	13 993	12 410	10 827
$\frac{1}{3}$	17 284	11 440	10 272	9 103	7 934	6 765
$\frac{1}{2}$	10 417	6 589	5 823	5 057	4 292	3 526
$\frac{2}{3}$	4 988	2 942	2 543	2 144	1 745	1 346
$\frac{5}{6}$	1 312	724	606	488	371	253
.....	.....	.....	.....	.....	.....	.....

8 Subdivisions of Half Arch.						
k.	Values of $\frac{1}{n}$					
	0	0.5	0.6	0.7	0.8	0.9
$\frac{1}{8}$	27 116	18 678	16 991	15 303	13 616	11 928
$\frac{1}{4}$	21 094	14 216	12 841	11 465	10 090	8 714
$\frac{3}{8}$	15 462	10 156	9 095	8 034	8 972	5 911
$\frac{1}{2}$	10 417	6 589	5 823	5 057	4 292	3 526
$\frac{5}{8}$	6 152	3 726	3 241	2 756	2 271	1 786
$\frac{3}{4}$	2 865	1 646	1 403	1 159	915	672
$\frac{7}{8}$	749	404	335	266	197	128
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....
.....	.....	.....	.....	.....	.....	.....

$$G_k = \frac{I_0}{l^3} \int m\,x \frac{d\,s}{I} \text{(whole span)}$$

TABLE 32.—VALUES OF CONSTANTS, A, B, F, AND D, FOR DIFFERENT VALUES OF $\frac{1}{n}$.

$\frac{1}{n}$.	$A \times 100\,000$.	$F \times 100\,000$.	$B \times 100\,000$.	$D \times 100\,000$.
0	100 000	66 667	$33\,333 + 20\,000\,\frac{b}{a}$	$20\,000 + 2\,857\,\frac{b}{a} + 11\,111\,\frac{b^2}{a^2}$
0.5	83 333	46 667	$23\,333 + 12\,857\,\frac{b}{a}$	$12\,857 + 17\,460\,\frac{b}{a} + 6\,566\,\frac{b^2}{a^2}$
0.6	80 000	42 667	$21\,333 + 11\,429\,\frac{b}{a}$	$11\,429 + 15\,238\,\frac{b}{a} + 5\,656\,\frac{b^2}{a^2}$
0.7	76 667	38 667	$19\,333 + 10\,000\,\frac{b}{a}$	$10\,000 + 13\,016\,\frac{b}{a} + 4\,747\,\frac{b^2}{a^2}$
0.8	73 333	34 667	$17\,333 + 8\,571\,\frac{b}{a}$	$8\,571 + 10\,794\,\frac{b}{a} + 3\,838\,\frac{b^2}{a^2}$
0.9	70 000	30 667	$15\,333 + 7\,143\,\frac{b}{a}$	$7\,143 + 8\,571\,\frac{b}{a} + 2\,929\,\frac{b^2}{a^2}$

TABLE 33.—VALUES OF L_k, C_k, AND G_k, $\times$ 100 000 FOR VALUES OF $\frac{1}{n}$.

TOTAL LOAD OF 1 LB. CONTINUOUS FROM CROWN TO SPRINGING LINE.				TOTAL LOAD OF 1 LB. CONTINUOUS OVER ONE-THIRD OF HALF-SPAN FROM CROWN.			
$\frac{1}{n}$.	$C_k \times 10^5$.	$G_k \times 10^5$.	$L_k \times 10^5$.	$\frac{1}{n}$.	$C_k \times 10^5$.	$G_k \times 10^5$.	$L_k \times 10^5$.
0	16 667	12 500	$10\,000 + 7\,143\,\frac{b}{a}$	0	35 185	25 154	$19\,465 + 13\,334\,\frac{b}{a}$
0.5	11 667	8 333	$6\,429 + 4\,365\,\frac{b}{a}$	0.5	25 453	17 236	$12\,798 + 8\,274\,\frac{b}{a}$
0.6	10 667	7 500	$5\,714 + 3\,809\,\frac{b}{a}$	0.6	23 506	15 652	$11\,465 + 7\,262\,\frac{b}{a}$
0.7	9 667	6 667	$5\,000 + 3\,254\,\frac{b}{a}$	0.7	21 560	14 069	$10\,131 + 6\,251\,\frac{b}{a}$
0.8	8 667	5 833	$4\,286 + 2\,698\,\frac{b}{a}$	0.8	19 613	12 485	$8\,798 + 5\,239\,\frac{b}{a}$
0.9	7 667	5 000	$3\,571 + 2\,143\,\frac{b}{a}$	0.9	17 667	10 901	$7\,464 + 4\,227\,\frac{b}{a}$

TABLE 34.—Powers of k for Different Half-Span Subdivisions.

20 Subdivisions.				11 Subdivisions.				9 Subdivisions.				8 Subdivisions.			
k.	k^2.	k^3.	k^4.	k.	k^2.	k^3.	k^4.	k.	k^2.	k^3.	k^4.	k.	k^2.	k^3.	k^4.
0	0	0	0	0.09091	0.00826	0.00075	0.00007	0.11111	0.01235	0.00137	0.00015	0.12500	0.01562	0.00195	0.00024
0.05	0.00250	0.00012	0.00001	0.18182	0.03306	0.00601	0.00109	0.22222	0.04938	0.01097	0.00244	0.25000	0.06250	0.01562	0.00391
0.10	0.01000	0.00100	0.00010	0.27273	0.07438	0.02029	0.00553	0.33333	0.11111	0.03704	0.01235	0.37500	0.14062	0.05273	0.01977
0.15	0.02250	0.00337	0.00051	0.36364	0.13223	0.04808	0.01748	0.44444	0.19753	0.08779	0.03902	0.50000	0.25000	0.12500	0.06250
0.20	0.04000	0.00800	0.00160	0.45455	0.20661	0.09391	0.04269	0.55556	0.30864	0.17147	0.09526	0.62500	0.39062	0.24414	0.15259
0.25	0.06250	0.01562	0.00391	0.54545	0.29752	0.16228	0.08852	0.66667	0.44444	0.29630	0.19753	0.75000	0.56250	0.42187	0.31641
0.30	0.09000	0.02700	0.00810	0.63636	0.40496	0.25770	0.16399	0.77778	0.60494	0.47051	0.36595	0.87500	0.76567	0.66992	0.58618
0.35	0.12250	0.04287	0.01501	0.72727	0.52893	0.38467	0.27976	0.88889	0.79012	0.70233	0.62430	.....	.....	.....	.....
0.40	0.16000	0.06400	0.02560	0.81818	0.66942	0.54771	0.44812	.....	.....	.....	.....	.....	.....	.....	.....
0.45	0.20250	0.09112	0.04101	0.90909	0.82645	0.75131	0.68301	.....	.....	.....	.....	.....	.....	.....	.....
0.50	0.25000	0.12500	0.06250												
0.55	0.30250	0.16637	0.09150												
0.60	0.36000	0.21600	0.12960	7 Subdivisions.				6 Subdivisions.							
0.65	0.42250	0.27462	0.17851												
0.70	0.49000	0.34300	0.24010												
0.75	0.56250	0.42187	0.31641	k.	k^2.	k^3.	k^4.	k.	k^2.	k^3.	k^4.				
0.80	0.64000	0.51200	0.40960												
0.85	0.72250	0.61412	0.52201												
0.90	0.81000	0.72900	0.65610												
0.95	0.90250	0.85737	0.81451	0.14286	0.02041	0.00291	0.00042	0.16667	0.02778	0.00463	0.00077				
1.00	1.00	1.00	1.00	0.28571	0.08163	0.02332	0.00666	0.33333	0.11111	0.03704	0.01235				
				0.42857	0.18367	0.07872	0.03374	0.50000	0.25000	0.12500	0.06250				
				0.57143	0.32653	0.18659	0.10662	0.66667	0.44444	0.29630	0.19753				
				0.71429	0.51020	0.36443	0.26031	0.83333	0.69444	0.57870	0.48225				
				0.85714	0.73469	0.62974	0.53977	.....	.....	.....	.....				

TABLE 35.—CURVES FOR ARCH AXIS. RATIO $\frac{y}{r}$ FOR VALUES OF $\frac{b}{a}$.

$\frac{b}{a}$	HORIZONTAL DISTANCE FROM CROWN IN FRACTIONS OF HALF-SPAN.								
	1/10	1/5	3/10	2/5	1/2	3/5	7/10	4/5	9/10
0	0.01	0.04	0.09	0.16	0.25	0.36	0.49	0.64	0.81
0.1	0.0091	0.0366	0.0826	0.1477	0.2329	0.3390	0.4673	0.6190	0.7960
0.2	0.0083	0.0336	0.0763	0.1376	0.2187	0.3216	0.4483	0.6016	0.7843
0.3	0.0077	0.0311	0.0711	0.1290	0.2067	0.3068	0.4323	0.5868	0.7745
0.4	0.0072	0.0290	0.0666	0.1216	0.1964	0.2942	0.4186	0.5742	0.7660
0.5	0.0067	0.0272	0.0627	0.1152	0.1875	0.2831	0.4067	0.5632	0.7587
0.6	0.0063	0.0256	0.0593	0.1096	0.1797	0.2735	0.3963	0.5536	0.7523
0.7	0.0059	0.0242	0.0563	0.1047	0.1728	0.2651	0.3871	0.5451	0.7466
0.8	0.0056	0.0229	0.0536	0.1003	0.1667	0.2575	0.3789	0.5376	0.7416
0.9	0.0053	0.0218	0.0512	0.0963	0.1612	0.2508	0.3716	0.5309	0.7371
1.0	0.0050	0.0208	0.0490	0.0928	0.1563	0.2448	0.3650	0.5248	0.7330

For filled spandrel arches, the factors for the odd twentieths, and for continuous loads covering the "half span", and one-third of the "half span" from the crown will be sufficient.

Open spandrel arches, however, are not so simple, since the span can be divided by columns or cross-walls into any number of parts. These tables provide for the division of the span by columns into any number of parts up to twelve, and any even number of parts up to twenty-two. Thus, they should cover all cases, provided there is no esthetic advantage in specifying that there should be no column at the crown, where the number of columns is large.

HARDY CROSS,* M. AM. SOC. C. E. (by letter).—The author has done a careful and scholarly piece of work in giving a method of approximating the stress coefficients and elastic properties of fixed masonry arches. The writer does not intend any criticism of the paper by his comments. He thinks, however, that, from the viewpoint of practical design, the mathematical theory of the right fixed arch is being overdone. If the abutments do not settle, spread, or rotate, the stresses which would exist in a simple beam are neutralized by stresses due to restraint; if the abutments spread without settlement or rotation the arch acts as a fixed-ended beam. The stresses produced by the loads on the structure must not be excessive, nor must the proportions of the structure be extravagant for its function. The loading, however, is somewhat problematic, the allowable stresses unknown, and there is a large "if" as to foundation conditions; having swallowed these "camels", only the "gnat" of the mathematical analysis remains. The "gnat" should be a *hors d'œuvre* and engineers are giving abnormal gustatory attention to it.

Three papers on elastic arches published by the Society within a year have analyzed a symmetrical right arch. None of them has discussed allowable fiber stresses, none has discussed the magnitude or probability of foundation failure. About the former little is accurately known. Reinforced concrete arches are not like simple beams, for bond and shear are small and the whole section is usually in compression; there seems good cause to think that they differ in many ways from columns, for several reasons besides the fact that it is scarcely

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conceivable that columns of the slenderness ratios of existing arches would carry their loads. The writer knows of no case of failure of a right hingeless bridge arch on good foundations. Apparently, they are, if the foundations hold, among the most reliable of all structures. No one knows accurately at what computed stresses they will fail; engineers are justified, therefore, in computing the stresses only with sufficient accuracy to show that there is no wide discrepancy between the computed stresses and those which are known to be safe in other types of structures; but on none of those structures by comparison with which the working stresses for arches are estimated, is there lavished the elaborate analysis which arches seem to demand.

The writer is strongly of the opinion that great precision in arch computations is not justified. In addition to the uncertainties enumerated, the theory of temperature stress is quite inadequate. The temperature changes may be considered to be made up of two parts; those changes occurring within, say, a month and those that are slow or seasonal. The Central States have a monthly variation of air temperature of about 60° Fahr.—perhaps 40° Fahr. in the concrete—superimposed on a slow seasonal variation of about 60° Fahr. in both air and concrete. The monthly variation is probably too rapid for the plastic yield of the concrete to affect the results materially; and no good reason appears why theoretical analysis is not a reasonable guide. For the seasonal changes, however, plastic yield equivalent to a reduction in elastic modulus to perhaps one-third its normal value is not improbable. Only more accurate laboratory data as to the plastic flow will make exact analysis possible; strain measurements in the field seem utterly useless in evaluating these phenomena, for if plastic flow occurs deformations can not in the light of existing data be quantitively interpreted.

Since plastic flow also probably affects the values of the stresses from dead load moments and from rib-shortening due to dead load, it is doubtful whether these stresses are very important.

Precise computations are especially futile in the case of spandrel filled arches and of short open web arches. In spandrel filled arches there is great uncertainty as to the distribution and direction of the pressure from the dead weight of the fill. Where the fill is deep, the uncertainties as to the distribution of live load by the fill, as well as within the ring itself, make it doubtful whether it is worth while to consider the live load at all except to include it as uniformly distributed dead load. In short open spandrel arches, the posts are usually sufficiently stiff, especially near the crown, to affect seriously the line of resistance. This source of error exists, of course, to some extent in all open web arches. Between expansion joints, the floor forms a top chord to a Vierendeel girder.

In passing, it may be remarked that Winkler's theorem, which fixes for constant section arches under vertical loads the true equilibrium polygon as the one lying nearest to the arch axis, as determined by least squares—or by the eye—is in danger of dropping out of arch literature. It is a direct consequence of the elastic theory and is perhaps the most important deduction from it. Except for the effect of rib-shortening under vertical loads, it is

rigidly correct for constant values of $\frac{d\,s}{I}$ and is not much in error for cases of variable $\frac{d\,s}{I}$.

In those cases, however, where more exact methods are demanded by the specifications, or by the judgment or taste of the designer, an accuracy of a few per cent. may be obtained by computing at most two cases of live load. For example, computations may be made for the live load extending three–eighths of the span from one abutment, and for the full live load, and from these all the maximum stresses may be derived. It is to be noted in any case that the "load divides" for the fiber stress are not coincident with those for the bending moment and these are further slightly modified by the effect of rib-shortening. If, then, any one considers that these assumed load divides are not sufficiently accurate, he must determine those for the moment at the kern points, the error due to assuming them as coincident with those for the axial moment being greater than is involved in the assumption just indicated. Really, it is absurd to quibble over whether a congested load stops short within a foot or two of a load divide.

While, therefore, a designer should be familiar with the shape of typical influence lines for arches, their exact computation for ordinary designs seems a waste of time. Even in the case of long spans, their value may be psychologic. As to how the stresses shall be found for the fixed loadings assumed, the writer expresses no preference here. He has used the constructions given by the author and strongly recommends them; but like every one else who has devoted much time to the study of arches, he has his own pet methods.

The theories of arch analysis which are now being elaborated in engineering literature are distinctly "high-brow" in that their elaborateness camouflages with erudition uncertainties and inaccuracies which are inevitable. Mathematically they all come to exactly the same thing. It is then the convenience rather than the accuracy of the author's curves which makes them valuable. It is not to be expected that any such curves will give great accuracy in practical design. It may perhaps be said that the results will be accurate if the arches are properly designed. This may be true as regards the theoretical curve of the arch axis and the theoretical variation of thickness, but in practice these matters are affected largely by the designer's judgment as to appearance of the structure and as to convenience of construction in the field.

The writer has been much interested in the problem of approximately generalizing the properties of arches, especially in his studies of continuous arches on elastic piers. In designing continuous arches it is desirable to know in advance the approximate elastic properties. These are, for symmetrical cases, the position of the horizontal neutral axis, the total elastic weight, and the two moments of inertia of elastic weights about the neutral axes. For a parabolic arch having a variation of I according to the secant:

$$y_c = \frac{1}{3\,r};\ W = \int \frac{d\,s}{E\,I} = \frac{l}{E\,I_c};\ I_v = \int x^2\,\frac{d\,s}{I} = \frac{1}{12}\,W\,l^2;$$

$$I_h = \int y^2 \frac{ds}{E\,I} = \frac{1}{11.25}\,W\,r^2$$

Mr. Whitney has tabulated values for y_c and I_h for his arches. His formulas:

$$W = \frac{l}{E\,I_c}\,\frac{1+m}{2} \qquad \text{.....(56)*}$$

and,

$$I_y = \frac{l^3}{12\,E\,I_c}\,\frac{1+3\,m}{4}\dagger$$

are also valuable.

Arch designs would be improved if a great deal more time were spent in studying the foundations at the expense of time devoted to the arch stresses. The author's paper has additional value if it encourages designers to apply to the advantage of the design the time saved by the use of his curves.

JAMISON VAWTER,‡ Assoc. M. Am. Soc. C. E. (by letter).—The writer has read Mr. Whitney's paper with considerable interest, particularly as to the use of the influence lines, the equation for the arch axis, and the formula for the variation of the rib thickness.

The form of the influence lines (Figs. 28 to 49) is applicable to existing arches, as the shape of the arch axis is taken into account by considering the ratio of the ordinate at the quarter-point to the rise. In fact, all the elements that will enter into the analysis of an arch are taken into account in these influence lines except the variation of thickness of the arch ring in the haunch. The writer believes that the variation of the rib thickness in the haunch of arches found in practice, as compared with the variation based on Mr. Whitney's formula, will not be sufficiently different to cause any material error in using these diagrams in connection with such arches, all other elements being the same.

These influence lines are much more applicable to the arches found in practice than those presented by Victor H. Cochrane, M. Am. Soc. C. E., in his paper entitled "The Design of Symmetrical Hingeless Concrete Arches".§ Such influence lines are used not only in the design of an arch the axis of which is determined by a given equation (such as that presented by Mr. Whitney), but also in the analysis of any other arch. With a given span, rise, and values of m and N, in general the arch will differ little from one of the author's with the same values of these elements. If, in addition, the value for ϕ_s for this arch is practically the same as the value obtained from Table 9, the arch is essentially the same as one obtained from Equation (38). In order that these influence lines may be of general value for this latter use, the values of m and N—particularly m—should cover a wider range.

The writer made a search through a few technical publications, taking only those arches that had sufficient data to determine values of all the neces-

* See p. 984.

† See p. 987.

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§ *Proceedings*, Eng. Soc. of W. Pa., Vol. 32 (February, 1916–January, 1917), pp. 670-682.

sary elements. Three of these six arches considered had values of m less than the minimum of 0.15 given by Mr. Whitney and one of these three had a value of N greater than 0.25. The lowest value of m was 0.054. Five of the arches had practically the same equation for their axes as the author's, the values of ϕ_s varying from his to the extent of from a few minutes to a little more than a degree. In the remaining arch the value of ϕ_s differed by 5° 10′ but the values of m and N fell within the range noted by the author. For a large number of arches there were insufficient data for exact computation, but inspection indicated that the number whose value of m would be less than 0.15 would probably be more than 40 per cent.

The writer made a further comparison with twenty arches given by M. A. Howe,* M. Am. Soc. C. E., that were apparently taken from practice. Twelve had values of m less than 0.15, one being as low as 0.02 and ten less than 0.10. Twelve had values of N greater than 0.25, the highest being 0.30; of these, six had values of both m and N outside the range noted in the paper, making a total of eighteen arches to which the influence lines presented by Mr. Whitney were not directly applicable.

The author takes 0.25 as the limit of N as this is the value for the parabola that the line of thrust under uniform loading would follow. In normal cases (where w_s is greater than w_c), the curve obtained from the funicular polygon would be above a parabolic curve, making N less than 0.25. In cases where w_s is less than w_c (if there were a high fill at the crown and open spandrel construction at the ends—a very uncommon type of construction), it is easily seen that the value of N would be greater than 0.25. However, it is possible to have a value of N greater than 0.25 if w_s is greater than w_c. As, for example, when the arch has a low spandrel fill over about the middle third or fourth and open spandrel construction at the ends (a not uncommon type of structure). The load at the crown will be greater than at the haunch, but less than at the springing line. The value of N will be more or less than 0.25, depending on the relation between these loads.

On an average, a ratio of 1:2 in the thickness of the arch rib at the crown and springing line will give a value of m of about 0.15. There are a large number of arches that have a greater thickness of rib at the springing line than this. The diagrams presented by Mr. Cochrane† provide for a ratio of 1 : 2.5 in the open spandrel type and 1:3 in the filled spandrel type. Thus, his curves have greater applicability than those of the author when used on ordinary arches. The writer does not believe that it is necessary, as far as stresses are concerned, to have a thickness at the springing line greater than twice that at the crown, in any normal case. Rough calculations on the six arches already noted indicate that even in those that have a greater ratio than this, economy of design could have held them in the range given by Mr. Whitney. Economy in the quantity of concrete that goes into the rib should not be the controlling feature in its design; the esthetics should be given the most weight in all except the most utilitarian structures. This will explain the chief reason for

* *Engineering News*, June 2, 1910.

† *Proceedings*, Eng. Soc. of W. Pa., Vol. 32, pp. 670–682.

the great depth of rib at the springing line in some arches, particularly those that have a curve for the intrados, that is elliptical, or nearly so.

The values of N, as found in practice, will rarely be much greater than 0.25 or 0.30. For that reason the author's influence lines, as given, can be used to advantage, interpolating beyond 0.25. On account of the large number of cases in which m will be much less than 0.15, it will be undesirable to interpolate beyond the author's diagrams to a smaller value of m; for that reason the writer would like to see these curves extended to provide for values of m as low as 0.05 so that these diagrams can be used to greater advantage, not only in the design of an arch according to the author's method of procedure, but also in the investigation of any arch.

In arches having a dead load of an irregular character, so that the author's equation for the axis is not directly applicable, the writer would use the influence lines in the preliminary design. With the span and rise known, he would draw the funicular polygon for the assumed dead loads plus one-half the full live load, and then the arch rib, causing the axis to fit this funicular polygon as closely as possible, and making approximate computations for d_c and d_s, allowing for the effect of temperature and rib-shortening. The values of N and ϕ_s could be taken from this arch rib and the approximate m calculated. If this ϕ_s should practically agree with that for the same span, rise, and N, as obtained from Table 9, the design based on the values obtained from the author's influence lines should be as close as desired. If the value of ϕ_s should vary from that given by the author by only a few degrees, the writer believes that the use of the influence lines would give values approximately correct. The thrust and moment due to temperature could be obtained from Fig. 56 and Table 17. It would be easier to compute the thrust due to rib-shortening separately; the writer believes that sufficiently close results can be obtained by dividing the thrust due to a fall in temperature of 1° by 12 (this being the product of the coefficient of thermal expansion and the modulus of elasticity of concrete) and multiplying by the average compression, in pounds per square inch, over the arch rib.

The writer advocates using the dead load plus one-half the full uniform live load in laying out the arch axis, as this loading will give a curve that will always decrease the maximum moment at the crown. If the arch axis is laid out according to this loading, the pressure line for the dead load alone will fall above the arch axis in the haunch and below it at the crown and springing line, giving a negative moment at these last two points. The author covers this point in the paper, but advocates using one-half the live load only when the rise is high and the effects of temperature and rib-shortening are slight. In this latter contingency the critical moments at both the crown and springing line will be positive, and it is easily seen that the use of a part of the uniform live load in laying out the arch axis will decrease these moments. In the case of a low rise ratio the critical moment at the crown will be positive and that at the springing line negative, and the loading advocated by the writer for laying out the axis will tend to decrease the critical moment at the crown and increase it at the springing line. An arch rib proportioned accord-

ing to the writer's ideas of what constitutes good appearance will have enough greater thickness at the springing line to reduce the stress at this point to an amount less than at the crown. For these reasons the writer believes that true economy, both in appearance and quantity of concrete in the rib, can best be effected by using a part of the full live load in laying out the arch axis, in all cases except possibly where the rise is very small, and this condition does not occur with sufficient frequency to justify making it an exception.

Charles S. Whitney,* M. Am. Soc. C. E. (by letter).—The writer wishes to correct any false impression as to his meaning in stating that the tables and diagrams given in the paper can be used without a thorough knowledge of the entire theory. The statement was perhaps incomplete and misleading. An engineer experienced in arch design can familiarize himself with the use of the tables and diagrams and with their limitations without following through all the mathematical process of derivation. The data in the paper can be used by one who is not an expert in arch analysis, provided the designing is done under the direction of an engineer who is thoroughly familiar with both the practical and theoretical aspects of arch design.

Mr. Harder is entirely correct in pointing out that the design of arches should not be entrusted to a novice. The writer's paper is limited to the mathematical considerations of design and does not attempt to treat the equally important practical considerations. There are many practical problems in arch design (such as those mentioned by Mr. Hayden and Mr. Harder) which must still be solved.

The propriety of publishing tables and diagrams that simplify engineering design has been questioned by some engineers because the apparent simplicity of their use has a tendency to encourage incompetent men to undertake designing. Although this is undoubtedly a fact to some extent, and is certainly to be regretted, encouragement should be given to the publication of material which will relieve the engineer of unnecessary labor in the details of design and permit him to direct his attention to the more important considerations. Modern construction operations are increasing in complexity rather than becoming more simple, and any standardization that can be effected without loss in efficiency and economy in the use of materials is advantageous. No tables or diagrams should be used without a thorough knowledge of their basis and limitations.

It has been the writer's purpose to present a method of design as opposed to a method of analysis. The former, of course, must include the latter. Means have been provided for studying the effect of varying the thickness of the rib at the crown and the springing line; but, in order that such a study may be made, it is first necessary to establish the arch axis and the law of variation of the thickness of the rib from the crown to the springing line. The only restriction placed on the designer is that he shall use the arch axis and the rib-thickness variation established by the formulas. The writer does not know of any other method for laying out the arch axis or for establishing

* Cons. Engr., Milwaukee, Wis.

the variation of rib thickness that will ordinarily give more satisfactory results from a standpoint of economy and appearance, and he believes that this restriction is rather beneficial than otherwise. No attempt should be made to standardize arch design generally because the number and range of variables is too great. There is, however, no advantage in using ribs with axes that depart greatly from the dead load pressure line or that have too great a thickness at the springing line as compared with the thickness at the crown.

The following more or less complete list of the variables that affect arch design is of interest in this connection:

(1) End conditions, rigidity of abutments.
(2) Length of span.
(3) Rise of axis.
(4) Allowable stresses in concrete and steel.
(5) Moduli of elasticity of concrete and steel.
(6) Coefficient of thermal expansion.
(7) Range of temperature.
(8) Amount of shrinkage in concrete.
(9) Live loading.
(10) Variation in thickness of rib from crown to springing line.
(11) Shape of arch axis.
(12) Thickness of rib at crown.
(13) Thickness of rib at springing line.
(14) Amount of reinforcement.
(15) Dead load.

Of these fifteen variables, the first nine are usually fixed by the specifications or assumed by the designer; the last six are interdependent and their proper values can be determined only by assuming simultaneous values and finding the resulting stresses. Whether or not the maximum stresses are excessive cannot be determined until all fifteen conditions are fixed. The fact that one assumed combination of values for Variables (10), (11), (12), (13), and (14), gives maximum stresses not exceeding the allowable, does not indicate that some other combination of values will not give a much superior and more economical arch. That is the reason for the inadequacy of that method of "design" which merely consists in analyzing the stresses in one assumed rib.

The method presented by the writer makes it possible to establish at once all but the last four quantities, and the designer is given facilities for making a complete study of the effect of varying values of the rib thickness at the crown and the springing line and of the amount of reinforcement. He is thus relieved of the most tedious mathematical work of stress analysis and is permitted to devote his energy to securing efficient proportions for the rib.

The writer wishes to express his appreciation of Professor Cain's discussion which has added so much to the value of the paper. Professor Cain states that he has checked all the formulas and has found them correct. He presents an excellent numerical method of analysis that requires less time

for him to apply than the graphical method. No doubt many other engineers will prefer his method. The writer prefers the graphical method because the curves appear to be easier to check and mean more to him than a table of numbers.

It is extremely interesting to note that the results of the three methods as applied to the same rib by Professor Cain and the writer agree closely. Professor Cain used the numerical method with equal $\frac{\Delta s}{I}$ divisions, while equal Δx divisions were used by the writer in the graphical solution. In each case, the span was divided into twenty sections.

Professor Cain has worked out a simple method of determining the reactions at the abutments produced by horizontal loads. He has kindly sent the derivation to the writer with the suggestion that it might be added to the discussion in order to make the treatment complete. The following is quoted from Professor Cain's letter:

"*Horizontal Load.*—When the arch axis has been divided into parts such that $\Delta w = \frac{\Delta s}{E I}$ is constant, then Equations (29), for $P = 1$, reduce to:

$$H_L = -\frac{\sum_A^R y'' y}{\sum_L^R y^2}; \quad V_L = \frac{\sum_A^R y'' x}{\sum_L^R x^2}; \quad M' = \frac{\sum_A^R y''}{N};$$

in which, N = the whole number of points (20 in Fig. 66).

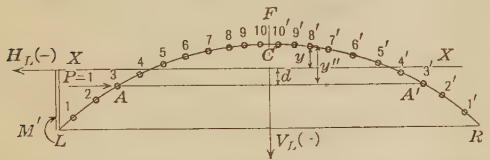

FIG. 66.

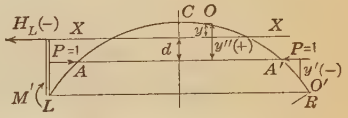

FIG. 67.

"The algebraic signs of x and y must be strictly attended to. From the method of locating the axis, XX,

$$\sum_L^C y = \sum_C^R y = 0.$$

In Fig. 66, d is + or — according as $P = 1$ acts below or above the axis, XX, therefore,

$$y'' = d + y.$$

"Let n = the number of points between L and A, where P acts. This does not include the point, A; thus, for $P = 1$ acting at 3, $n = 2$.

$$\sum_A^R y'' y = \sum_A^R (d + y) y = d \left[\sum_L^R y - (y_1 + y_2 + \ldots + y_n) \right]$$

$$+ \sum_L^R y^2 - (y_1^2 + y_2^2 + \ldots + y_n^2).$$

Hence, noting that $\sum_L^R y = 0$,

$$H_L = -\left\{1 - \frac{(y_1^2 + y_2^2 + \cdot\cdot + y_n^2) + d\,(y_1 + y_2 + \cdot\cdot + y_n)}{\sum_L^R y^2}\right\}.$$

"To reduce the formula for V_L, note from Fig. 66, since x is positive from C to R, and negative from C to L, that

$$\sum_A^R x = \sum_{A'}^R x + \sum_{A'}^C x + \sum_A^C x = \sum_{A'}^R x = x_1 + x_2 + \cdot\cdot x_n$$

$$\sum_A^R x\,y = \sum_{A'}^R x\,y + \sum_{A'}^C x\,y + \sum_A^C x\,y$$

$$= \sum_{A'}^R x\,y = x_1\,y_1 + x_2\,y_2 + \cdot\cdot + x_n\,y_n$$

Then,

$$\sum_A^R y''\,x = \sum_A^R (d + y)\,x = d\sum_A^R x + \sum_A^R x\,y$$

and,

$$V_L = \frac{d\,(x_1 + x_2 + \cdot\cdot + x_n) + (x_1\,y_1 + x_2\,y_2 + \cdot\cdot + x_n\,y_n)}{\sum_L^R x^2}.$$

"Again, since $\sum_L^R y = 0$,

$$\sum_A^R y'' = \sum_A^R (d + y) = (N - n)\,d + \sum_L^R y - (y_1 + y_2 + \cdot\cdot + y_n),$$

and,

$$M' = \frac{\sum_A^R y''}{N} = \frac{(N - n)\,d - (y_1 + y_2 + \cdot\cdot + y_n)}{N}.$$

"*Two Symmetrical Horizontal Loads.*—The case of two horizontal loads, $P = 1$, both acting inward, is shown in Fig. 67. The arch is supposed to be fixed at the right end and free at the left end.

"With O anywhere on the axis from A to R, the moment due to $P = 1$ at A is, $-P\,y''$. With O' between A' and R, y'' at O' is negative, therefore, the moment at O', due to $P = 1$ acting to the left at A', is $P\,y''$. The formula for H_L is now written:

$$H_L = \frac{-\sum_A^R y''\,y + \sum_{A'}^R y''\,y}{\sum_L^R y^2}$$

$$= \frac{-\sum_A^{A'} y''\,y - \sum_{A'}^R y''\,y + \sum_{A'}^R y''\,y}{\sum_L^R y^2}.$$

Then,

$$H_L = -\frac{2\sum_A^C y'' y}{\sum y^2}.$$

$$\sum_A^C y'' y = \sum_A^C (d y + y^2) = \sum_L^C (d y + y^2) - \sum_L^A (d y + y^2)$$
$$= \sum_L^C y^2 - \sum_L^A (d y + y^2).$$

Therefore,

$$2\sum_A^C y'' y = \sum_L^R y^2 - 2\left(\sum_L^A d y + \sum_L^A y^2\right)$$

and,

$$H_L = -\left[1 - 2\frac{(y_1^2 + y_2^2 + .. + y_n^2) + d(y_1 + y_2 + .. + y_n)}{\sum_L^R y^2}\right]$$

"In the numeration of the formula for M',

$$\sum_A^R y'' - \sum_{A'}^R y'' = \sum_A^{A'} y'' + \sum_{A'}^R y'' - \sum_{A'}^R y''$$

Therefore,

$$M' = \frac{2\sum_A^C y''}{N}.$$

$$\sum_A^C y'' = \sum_L^C (d + y) - \sum_L^A (d + y)$$
$$= \sum_L^C d - \sum_L^A (d + y)$$

Then,

$$M' = d - \frac{2[n\,d + (y_1 + y_2 + .. + y_n)]}{N}$$

"The value of V_L is zero, since the part of it due to P at A, acting downward, is exactly balanced by the part due to P at A' which acts upward.

"The numerical values given by the formulas for one horizontal load check within 2% of the values given by the graphical method.

"The variation of the bending moment along the rib for the two cases has been accurately shown by Howe."*

Professor Cain has remarked that for use in laying out the rib axis a table showing the co-ordinates of twenty points on the axis of the half rib would be desirable. The writer believes that ten points are sufficient for any but very long spans if the tangents at each point are located by the method given.

If more points are desired, they may easily be located by the use of the formula instead of the table.

The method of analysis given by Professor Martel is a valuable addition to the discussion. The writer believes that the analysis of all important arches should be checked by some method that takes into consideration the exact

* "Symmetrical Masonry Arches," Figs. 10 and 11, pp. 39 and 41.

dimensions and reinforcement of the rib as finally designed. Although the tables and diagrams given in the paper should ordinarily be satisfactory for establishing an economical design for a large arch, the final design should be checked back by an independent method to eliminate errors from any source.

Mr. Harder's discussion is particularly interesting because it comes from an engineer who is already familiar with the method given in the paper and is experienced in arch design. It is his opinion that the arch axis should follow the pressure line of the dead load as closely as possible. In the case of an open spandrel arch, the two curves cannot, of course, be made to conform exactly because the pressure line is broken at the load concentrations while the arch axis should be a smooth curve. In such a case, the concentrated loads may be considered as distributed for the purpose of locating the arch axis; but in computing the stresses the effect of the concentrations must be taken into account.

The term "transformed catenary" applies not only to the curve given by Equation (38) but also to a group of similar curves which vary in their suitability for use as the arch axis. The curve given by Equation (38) for the arch axis has been found satisfactory not only for spandrel filled arches but also for open spandrel arches of long span. In all cases, the amount of variation between the true pressure line and the assumed axis should be determined during the process of design, and the shape of the axis may be corrected if that is advantageous.

An excellent method of determining the curve of the axis has been used by Freyssinet in designing several long-span arch ribs. A curve was plotted giving the actual dead load per unit length of span for each point in the span. Through four points on this curve was passed a parabola of the fourth degree, which was close to the actual load curve. By double integration of the fourth degree parabola, a sixth degree parabola was obtained which represented the pressure line of the loads given by the fourth degree curve. This sixth degree curve was made to pass through the crown and springing points and was taken as the arch axis. The slight differences between the actual load line and the fourth degree parabola were treated as isolated loads to determine the dead load bending moments.

The equation of the rib axis found in this manner for the Villeneuve-sur-Lot Bridge was:

$$y = 0.47619\,\frac{x^2}{10}\left(1 + 0.00681819\,\frac{x^2}{10} + 0.0000161006\,\frac{x^4}{10}\right)$$

The span was 96.25 m.

Using the method given in the paper, the writer would suggest the following procedure for important arches. After designing the rib by the tables and diagrams, compute the actual dead loads and determine the true pressure line for these loads, assuming it to pass through the crown and springing points. Compute the value of N by dividing the ordinate of this pressure line at the quarter-point of the span (measured from the horizontal through the crown) by the rise, r. With this value of N, compute the value of g from Equation (41). Using this value of g, the arch axis curve can then be obtained

from Equation (38), or Table 6. This axis curve will coincide with the pressure line at the crown, quarter-points, and springing line and will lie close to it between those points. In order to take into consideration any divergency between the axis and the true dead load pressure line, the dead load bending moments should be computed by means of the influence lines.

In Fig. 63 Mr. Harder shows the composition and distribution of the maximum positive and negative bending moments for a typical arch design. He points out the impossibility of designing a rib in which the maximum stresses would be uniform from crown to springing line. The writer believes that the variation in thickness of rib as given in the paper is quite satisfactory. As Mr. Harder states, this is based on the general expression:

$$\frac{I_c}{I_x \cos \phi} = 1 - (1 - m)z^v$$

in which the value of v is assumed to be 1. Ritter has used the same expression with $v = 2$.* The bending moments shown in Fig. 63 were computed with $v = 1$. If $v = 2$, the rib would be thinner in the haunches and the positive live load moments at the crown would be increased while the temperature moments would be decreased. For ribs of moderate rise, the combined moments without temperature effect may control and the required crown thickness will then be slightly greater when $v = 2$ than when $v = 1$. This may offset the saving in material effected by using the thinner haunch.

The method given in the paper results in ribs that are not too thin in the haunch and are satisfactory in appearance. The rule mentioned by Professor Andrews, which makes the vertical projection of all radial sections equal, does not permit a variation in thickness at the springing line when the crown thickness is established and may result in ribs which are much too thick in the haunches. It cannot be applied to arches of high rise. It was quite common practice to determine the thickness of stone masonry arches by that rule as far as the point where $\phi = 60°$, beyond which it was modified to avoid excessive thickness. The writer has known of many arches which had too great a thickness at the springing line and in the haunches. The addition of this material is not only uneconomical but it increases the stresses due to temperature change and shrinkage and may even cause cracking.

In regard to the method of computing the thickness of the rib the opinions of Mr. Harder and the writer differ slightly. The writer has the latest edition of Strassner's book and has used the formulas which Mr. Harder gives for determining the rib thickness. Properly used, with a knowledge of their limitations, they give excellent results except when the ratio of rise to span is small and when the amount of reinforcement is great. They are based on assumed values of m and, therefore, may require as many trials as the writer's method when the temperature and rib-shortening effects are important. Unless the designer has some practice in the application of these formulas, the more direct method of first determining the total thrust and moment and then computing the stresses or the thickness required will be simpler. Diagrams

* "Beitrage zur Theorie und Berechnung der vollwandigen Bogentraeger ohne Sheitelgelenk", by Max Ritter.

have also been prepared which facilitate this operation, but owing to the length of the paper they have not been included.

The writer has not translated Strassner's work but has attempted to assimilate it and to present the method in combination with his own ideas in a suitable form for American practice. Technical literature has not covered the subject of arch design as well in this country as it has in some others. One monumental French work, to which the writer has never seen a reference in English, is "Grandes Voutes", by Paul Séjourné, Chief Engineer des Ponts et Chaussées and Chief Engineer of Construction of the Paris-Lyon-Mediterranean Railroad. This work is in six large volumes and covers the details of design and construction of most of the large arch bridges in masonry and of many in concrete built in all countries to 1916. It gives historical and architectural details as well as details of design.

In the case of unusually long and slender arch ribs, it may become advisable to determine the effect of wind pressure and lateral forces. A method of determining the effect of forces acting normal to the plane of the arch rib is given by Dr. J. Melan.* This analysis involves the behavior of the rib in torsion and the results are therefore rather uncertain if the width of the rib is great in proportion to the depth.

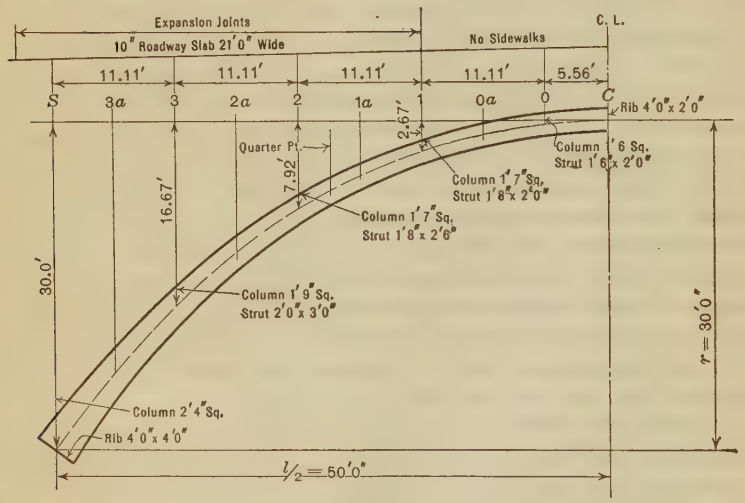

FIG. 68.

Mr. Warrington's discussion has raised some questions in regard to the manner of applying the formulas to open spandrel arches. Mr. Warrington has kindly sent additional data regarding the spans to which he referred in his discussion so that the cause of the discrepancies noted could be determined. The 100-ft. span only will be considered here as the same principles apply to the 94-ft. span. The construction of the 100-ft. span is as indicated in Fig. 68.

* "Handbuch für Eisenbetonbau", Third Edition, Vol. I, p. 734. For a more complete investigation, see "Forscherarbeiten auf dem Gebiete des Eisenbetons", Vol. XXVII, by Alfred Hauranek.

Mr. Warrington's computation of the dead loads is summarized in Table 36.

TABLE 36.—Dead Loads, in Pounds, for 100-Foot Span (One Rib).

Point.	Crown.	1.	2.	3.	Springing line.
Arch rib	14 100	15 400	17 200	25 600	46 400
Roadway	23 600	23 600	23 600	23 600	23 600
Floor-beam	3 000	5 200	3 000	3 000	5 200
Bracket	800	800	800	800	800
Columns	2 200	3 200	5 200	9 900	28 600
Strut	2 700	3 000	3 800	5 400	.....
Total per panel	46 400	51 200	53 600	68 300	104 600
Total per linear foot	4 180	4 610	4 820	6 150	9 410

There are several points in regard to the calculation of these loads which are typical of open spandrel designs and which perhaps should have been more fully explained in the paper. The arch rib is a distributed load; therefore, in computing, the section at the point considered should be used instead of the average section for the panel. The latter method, which Mr. Warrington used, gives a value slightly too high. All the other loads are concentrated at the columns and for them the total load per panel is divided by the panel length.

The expansion-joint floor-beams at Points 1 and at the springing line are heavier than the typical beam. This will produce an irregularity in the true pressure line, which it may be best to neglect in laying out the arch axis. Unless it is so large as to produce high stresses, this excess in weight of the expansion floor-beam over that of the typical beam (2 200 lb. in this case) can best be treated as an isolated load and the bending produced by it determined by use of the influence lines.

It may also be noted that the columns increase in weight toward the springing line but that the column at the springing line is disproportionately large so that a sudden increase in weight occurs which is not consistent with the rate of increase of the columns on the span. In this case, if the weights of the columns are plotted under their respective positions and a smooth curve is drawn through these points, a weight at the springing line of about 18 000 lb. is indicated. This should be used instead of the actual weight in computing the value of w_s. If all the columns had been the same in section or if they had increased uniformly in section to the springing line, this adjustment would not have been necessary. The actual column size here increased from 21 in. at Point 3 to 28 in. at the springing line.

Since there is a strut between the arch ribs at each column point on the span, it is necessary to assume that there is one at the springing line corresponding in size to those on the span. These struts increase in size toward the springing line; and, if the same law of variation is maintained, the strut at the springing line would weigh about 7 800 lb.

The theoretical w_s for open spandrel arches will seldom be the same as the actual weight. If the values of w have been computed for all the panel points

of the span, the theoretical value of w_s could be obtained by plotting the value of w at each panel point and continuing the curve to the springing line. This should be a smooth continuous curve, and the intercept at the springing line would give a value of w_s consistent with the weights on the span.

Correcting the weights of rib, columns, floor-beams, and struts given in Table 29, the weights at the crown and springing line would be about as given in Table 37.

TABLE 37.—Corrected Dead Loads, in Pounds, for Crown and Springing Line.

Point.	Crown.	Springing line.
Arch rib	13 330	43 400
Roadway	23 600	23 600
Floor-beam	3 000	3 000
Bracket	800	800
Column	2 100	18 000
Strut	2 700	7 800
Total per panel	45 530	96 600
Total per linear foot	4 100	8 690

From these loads, the value of g is computed:

$$g = \frac{w_s}{w_c} = \frac{8\ 690}{4\ 100} = 2.12$$

The corresponding value of N is (from Table 6) $N = 0.2224$, and the ordinate at the quarter-point is:

$$30 \times 0.2224 = 6.672 \text{ ft.}$$

The ordinate of the dead load pressure line at the quarter-point may be determined approximately by taking moments.

By moments about the springing line:

$$44.44 \times 46\ 400 = 2\ 063\ 000 \text{ ft-lb.}$$
$$33.33 \times 49\ 000 = 1\ 630\ 000 \text{ "}$$
$$22.22 \times 53\ 600 = 1\ 187\ 000 \text{ "}$$
$$11.11 \times 68\ 300 = 758\ 000 \text{ "}$$
$$2.78 \times 21\ 900 = 60\ 800 \text{ "}$$

$$\text{Total} = 5\ 698\ 800 \text{ ft-lb.}$$

The value for the horizontal thrust is, therefore:

$$H = \frac{5\ 698\ 800}{30} = 190\ 000 \text{ lb}$$

Taking moments about the quarter-point:

$$19.44 \times 46\ 400 = 903\ 000 \text{ ft-lb.}$$
$$8.33 \times 49\ 000 = 408\ 000 \text{ "}$$

$$\text{Total} = 1\ 311\ 000 \text{ ft-lb.}$$

The ordinate of the true pressure line at the quarter-point of the span is then approximately:

$$y_{\frac{1}{4}} = \frac{1\,311\,000}{190\,000} = 6.90 \text{ ft.}$$

If the arch axis is passed through this point, the value of N will be:

$$N = \frac{6.90}{30} = 0.230$$

and from Table 6 this corresponds to $g = 1.756$, which would make the value of w_s:

$$w_s = g\, w_c = 1.756 \times 4\,100 = 7\,200 \text{ lb.}$$

In order to determine how well the curve given by Equation (38) would correspond with the dead load pressure line for this arch, the writer has computed the ordinates to both curves using $N = 0.230$ and $g = 1.756$ for the former. These ordinates are obtained for the column points and for the points midway between the columns and are given in Table 38.

TABLE 38.—Comparison of Axis Curve and Dead Load Pressure Line.

Point. (1)	Ordinate of dead load pressure line. (2)	Ordinate of axis curve, (by Equation (38)). (3)
Crown	0.000	0.00
0a	1.357	1.33
1	2.825	3.02
1a	5.50	5.42
2	8.42	8.57
2a	12.64	12.54
3	17.17	17.43
3a	23.35	23.15
Springing line	30.00	30.00

It is seen from Table 38 that at the points where concentrated loads are applied, the pressure curve lies above the axis curve while midway between the columns the pressure line is below the axis curve. The axis is, therefore, following the position which the pressure line would take if the loads were distributed instead of concentrated at columns and this position is as near as any smooth curve can come to the actual pressure line. It should be noted that in determining the correct value of N, the ordinate of the pressure line for loads assumed as distributed should be taken. In this example it just happened that the quarter-point of the span fell about at the quarter-point of a panel between columns and the ordinate of the true pressure line for concentrated loads was used without great error.

The actual average loads per linear foot of rib at the column points are compared in Table 39 with the theoretical loads for which the proposed axis curve given in Column (3) of Table 38 is the pressure line.

This is shown graphically by Fig. 69. It is seen that the actual and theoretical loads on the span agree fairly well, and, of course, the load on

the abutment has no effect on the arch rib. The horizontal thrust computed from Table 9 for $N = 0.230$ is:

$$H = 0.1397 \frac{w_c l^2}{r} = 0.1397 \times 4\,100 \times \frac{10\,000}{30} = 191\,000 \text{ lb.}$$

This agrees well with the value of 190 000 lb. computed from the actual loading.

TABLE 39.—Comparison of Actual and Theoretical Loads.

Point.	Actual load, in pounds.	Load assumed by Equation (38), $N = 0.23$, in pounds.
Crown	4 100	4 100
1	4 410	4 422
2	4 820	5 015
3	6 150	5 960
Springing line	8 690	7 209

It appears from these calculations that Equation (38) gives a satisfactory form for the axis of an open spandrel arch when the proper value of g is used. This example given by Mr. Warrington is extreme in several respects. The rise is high and the rib is quite thick at the springing line. The dead load increases with unusual rapidity toward the springing line on account of the rapid increase in weight of the columns, struts, and arch ribs.

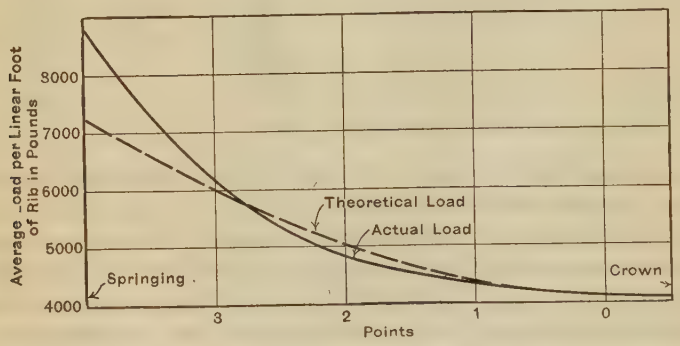

Fig. 69.—Graphic Comparison of Actual and Theoretical Loads.

The formulas have been applied to an open spandrel arch of 261-ft. span with a rise of 83.66 ft. $\left(\frac{r}{l} = 0.32\right)$, with satisfactory results. In this case, the actual value of $\frac{w_s}{w_c}$ was computed to be 3.18, which gave a quarter-point ordinate of 17.19 ft. The actual ordinate of the dead load pressure line was found to be 17.72 ft., corresponding to a value of g equal to 2.705 and this value was used in the analysis. With this adjustment, the axis given by

Equation (38) was found to correspond closely with the dead load pressure line.

Using the tables and diagrams of the paper, the writer has made an analysis of an open spandrel bridge design given by J. A. L. Waddell, M. Am. Soc. C. E.* These computations are given here to show how closely the results check those obtained by other methods and to explain a number of points which must be considered in open spandrel arch design. Fig. 70 gives a sketch of this span. The floor system is divided into ten slab spans of 9.33 ft. each, carried on cross-girders and cantilevers on spandrel columns resting on the two separate arch ribs.

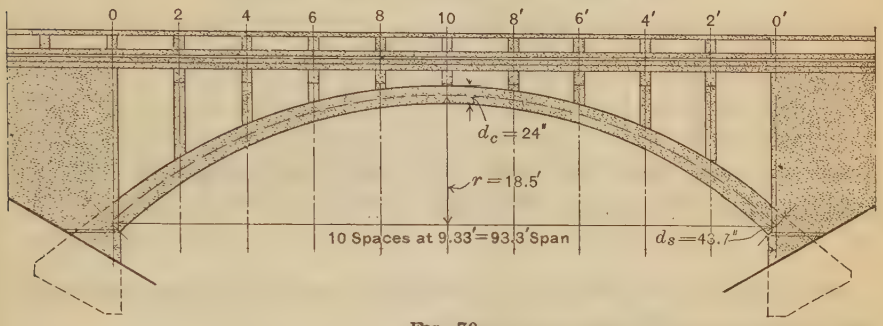

FIG. 70.

The dimensions of the span are as follows:

$l = 93.3$ ft.
$r = 18.5$ ft.
$b = 4$ ft. 6 in. gross for each rib.
$= 4$ ft. 4 in. for weights, allowing for paneling.
$= 4$ ft. 5 in. for moment of inertia.
$d_c = 24$ in.
$d_s = 43.7$ in.

The value of w_c and w_s will be computed first in order to establish the ordinates of the arch axis. This is done as if the load were distributed along the ribs, and it is found that w_c equals 4 675 lb. and w_s equals 7 490 lb. Then,

$$g = \frac{w_s}{w_c} = \frac{7\,490}{4\,675} = 1.604$$

From Table 9, with $g = 1.604$:

$$N = 0.23356$$
$$C_d = 0.1369$$
$$\frac{V_d}{w_c\, l} = 0.5970$$
$$\frac{l}{r} \tan \phi_s = 4.361$$

* "Bridge Engineering", Vol. I.

Then,

$$\tan \phi_s = 4.361 \times \frac{18.5}{93.3} = 0.864 \text{ (Waddell gives 0.865)}$$

and,

$$\sec \phi_s = 1.322 \text{ (Waddell gives 1.322)}$$

The ordinates of the arch axis (Table 40) may be computed from Table 6 by interpolation. These figures show that the arch axis given by Table 6 coincides almost exactly with that given by Waddell.

TABLE 40.—Ordinates of Arch Axis.

	Point.										
	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
$\frac{y_0}{r}$ (Table 6)..	1.00	0.7965	0.6196	0.4681	0.3398	0.2336	0.1483	0.0829	0.0367	0.0092	00
y_0, in feet....	18.50	14.75	11.45	8.65	6.28	4.32	2.74	1.53	0.68	0.17	00
y_0 (Waddell).	18.50	.....	11.46	.....	6.28	.....	2.78	.....	0.68	.....	00

The variation in the thickness of the rib will next be computed so that the dead weight of the rib can be obtained:

$$d_c = 24 \text{ in.}$$
$$d_s = 43.7 \text{ in.}$$

Hence,

$$m = \frac{d_c^3}{d_s^3 \cos \phi_s} = \frac{24^3}{43.7^3 \times 0.756} = 0.220$$

From Equation (49),

$$d_x = d_c\, c \sqrt[6]{1 + \tan^2 \phi}$$

The values of c and $\tan^2 \phi$ are to be obtained from Tables 12 and 13. The solution is given in Table 41, from which it is seen that the thickness given by Equation (49) is somewhat less in the haunch than that given by Waddell.

TABLE 41.—Thickness of Arch Rib.

Point.	$\frac{l^2}{r^2} \tan^2 \phi$. (Table 12)	$1 + \tan^2 \phi$.	$\sqrt[6]{1 + \tan^2 \phi}$.	c (Table 13).	d_x in inches.	d_x (Waddell), in inches.
0	19.030	1.750	1.098	1.661	43.7	43.70
1	14.439	1.568	1.078	1.499	38.8	
2	10.745	1.423	1.061	1.386	35.3	38.22
3	7.802	1.307	1.046	1.302	32.7	
4	5.467	1.215	1.033	1.235	30.6	33.86
5	3.653	1.144	1.023	1.180	29.0	
6	2.263	1.089	1.014	1.122	27.3	30.22
7	1.241	1.049	1.008	1.086	26.3	
8	0.543	1.021	1.008	1.059	25.5	27.02
9	0.135	1.005	1.001	1.027	24.7	
10	0.000	1.000	1.000	1.000	24.0	24.00

In regard to this, Waddell says:*

"A comparison of the stresses at the various sections indicates, as did the preliminary design, that the rib should have been made somewhat thinner in the haunches, thus effecting a saving in both concrete and steel amounting to several per cent."

This would show that the thickness given by Equation (49) is satisfactory for this particular arch.

Using the weights of floor system as computed by Waddell, the dead panel loads are, as follows:

At	Point	10	44 950	lb.	per rib.
"	"	8	44 350	"	" "
"	"	6	46 800	"	" "
"	"	4	51 250	"	" "
"	"	2	58 800	"	" "

The dead load thrusts and moments can now be determined from the influence lines. Using the same reinforcement as that given by Waddell, the moments of inertia for one rib are:

$$I_c = 3.72 \text{ ft.}^4$$
$$I_s = 23.39 \text{ ft.}^4$$

and,

$$m = \frac{I_c}{I_s \cos \phi_s} = \frac{3.72}{23.39 \times 0.756} = 0.210$$

The value of N was found to be 0.2336. Entering the influence line diagrams with these values of m and N, the horizontal thrust due to dead load is as given in Table 42.

TABLE 42.—Dead Load, H.

Point.	$\frac{Hr}{Wl}$	From Influence Lines.			Waddell.		
		$\frac{H}{W}$	Dead load, W, in pounds.	Dead load, H, in pounds.	$\frac{H}{W}$	Dead load, W, in pounds.	Dead load, H, in pounds.
0	0	0	.....	0 000	.....	.....	0 000
2	0.0230	0.116	58 800	6 820	0.120	60 800	7 300
4	0.0852	0.430	51 250	22 000	0.434	53 300	23 150
6	0.1660	0.837	46 800	39 100	0.831	48 500	40 300
8	0.2337	1.178	44 350	52 200	1.175	45 300	53 300
10	0.2608	1.316	44 950	59 200	1.322	45 500	60 100
Summation for span	.....		H = 299 440			H = 308 200	

The dead load vertical reaction at the springing line is equal to one-half the dead load on the span from Points 2 to 2′, inclusive, plus the weight of the rib from the springing to Point 1, or:

$$V_d = 223\,675 + 13\,460 = 237\,135 \text{ lb.}$$

* "Bridge Engineering", Vol. I, p. 909.

For the condition of distributed loading, Table 9 would give for H_d and V_d:

$$H_d = C_d \frac{w_c l^2}{r} = 0.1369 \times \frac{0.4675 \times 93.3^2}{18.5} = 301\ 500 \text{ lb.}$$

$$V_d = 0.5970\ w_c l = 0.5970 \times 4\,675 \times 93.3 = 260\ 000 \text{ lb.}$$

This value of V_d is too high because, for the concentrated load condition, the panel load at the springing line is carried by the column directly to the abutment and does not affect the rib. Subtracting one-half this column load (not including the weight of the rib from 0 to 1) from V_d:

$$V_d = 260\ 000 - (4.66 \times 4\,410) = 239\ 500 \text{ lb.}$$

This value agrees closely with the value computed from the influence lines. The value of 301 500 lb. for H_d obtained from Table 9 agrees closely with the influence-line value without correction because the loading near the end has little effect on the value of H. The difference between these values and those given by Waddell are, of course, almost entirely due to the difference in the thickness of the rib.

For the value of the ordinate, y_c, the distance from the elastic center of gravity of the rib to the rib axis at the center of the span, Table 17 gives for $N = 0.2336$ and $m = 0.21$:

$$\frac{y_c}{r} = 0.2147$$

$$y_c = 0.2147 \times 18.5 = 3.97 \text{ ft.}$$

Waddell gives $y_c = 3.851$ ft., the difference being due to the thicker haunch which raises the elastic center of gravity.

The effect of temperature change is obtained as follows: From Fig. 56:

$$C_t = 2.29$$

The temperature thrust for a drop of 50° Fahr. is, therefore:

$$H_t = C_t \frac{t\, I_c}{r^2} = 2.29 \times \frac{-50 \times 3.72 \times 12^4}{18.5 \times 18.5}$$

$$= -25\ 800 \text{ lb. (Waddell gives 29 700 lb.)}$$

The temperature moments are:

At the crown ..... $M_t = 25\ 800 \times 3.97 = 102\ 400$ ft-lb.

At the springing line .... $M_t = 25\ 800 \times (3.97 - 18.5) = -375\ 000$ ft-lb.

It is seen that the effect of the temperature change has been reduced considerably by using a rib that is thinner between the crown and the springing line.

The rib-shortening thrust is:

$$H_{RS} = -H \frac{u'}{1+u}$$

in which,

$$u = \frac{I_c}{A\ C_m\ C\ r^2} \text{ and } u' = \frac{I_c}{A\ C'_m\ C\ r^2}$$

From Table 18:

$$C = 0.0363$$

From Fig. 33, for $m = 0.21$ and $\frac{r}{l} = 0.1983$:

$$C_m = 1.324 \quad \text{and} \quad C_m' = 1.139$$

Then,

$$u = \frac{3.72 \times 12^4}{1.14 \times 24 \times 53 \times 1.324 \times 0.0363 \times 222 \times 222} = 0.02245$$

Similarly,

$$u' = 0.0261 \text{ and } \frac{u'}{1+u} = 0.0254$$

Therefore,

$$H_{RS} = -0.0254\ H \text{ (Waddell finds } H_{RS} = -0.0236\ H).$$

For dead load alone:

$$H_{RS} = -0.0254 \times 299\,440 = -7\,610 \text{ lb.}$$

The values of H, V, and M, at the crown and the springing line for unit loads at panel points taken from the influence line diagrams are given in Tables 43 and 44 and compared with the corresponding values found by Waddell.

TABLE 43.—Comparison of Live Load Thrusts and Moments at the Crown.

Point loaded.	From Influence Lines.			Waddell.		
	H.	*V.*	*M.*	*H.*	*V.*	*M.*
2	0.116	−0.018	−0.25	0.120	−0.019	−0.23
4	0.430	−0.079	−0.74	0.434	−0.081	−0.69
6	0.837	−0.185	−0.76	0.831	−0.181	−0.70
8	1.178	−0.330	+0.51	1.175	−0.331	+0.49
10	1.316	+0.500	+3.89	1.322	+0.500	+3.79
8′	1.178	+0.330	+0.51	1.175	+0.331	+0.49
6′	0.837	+0.185	−0.76	0.831	+0.187	−0.70
4′	0.430	+0 079	−0.74	0.434	+0.081	−0.69
2′	0.116	+0.018	−0.25	0.120	+0.019	−0.23
For $\Sigma + M$	3.672	+0.500	+4.91	3.672	+0.500	+4.77
For $\Sigma - M$	2.766	+0.000	−3.50	2.770	+0.000	−3.24
For ΣM	6.438	+0.500	+1.41	6.442	+0.500	+1.53

The values in these tables are a good check on the agreement between the results obtained by the two different methods.

The positive live load bending moment at the crown may now be computed. The live panel load is taken at 20 000 lb. per rib, which is approximately 2 120 lb. per lin. ft.:

$$M_c = 4.91 \times 20\,000 = 98\,200 \text{ ft-lb. (Waddell gives 95 000)}$$

The negative live load moment at the springing line is:

$$M_s = -19.09 \times 20\,000 = -381\,800 \text{ ft-lb. (Waddell gives } -372\,000).$$

TABLE 44.—Comparison of Live Load Thrusts and Moments at the Springing Line.

Point loaded.	From Influence Lines.			Waddell.		
	H.	*V.*	*M.*	*H.*	*V.*	*M.*
2	0.116	0.982	— 6.67	0.120	0.981	— 6.45
4	0.430	0.921	— 7.75	0.434	0.919	— 7.54
6	0.837	0.815	— 4.67	0.831	0.813	— 4.59
8	1.178	0.670	+ 0.39	1.175	0.669	+ 0.35
10	1.316	0.500	+ 4.90	1 322	0.500	+ 4.92
8′	1.178	0.330	+ 6.89	1.175	0.331	+ 6.78
6′	0.837	0.185	+ 6.11	0.831	0.187	+ 5.94
4′	0.430	0.079	+ 3.59	0.434	0.081	+ 3.56
2′	0.116	0.018	+ 1.03	0.120	0.019	+ 1.10
For $\Sigma + M$	5.055	1.782	+22.91	5.057	1.787	+22.65
For $\Sigma - M$	1.383	2.718	—19.09	1.385	2.713	—18.58
For ΣM	6.438	4.500	+ 3.82	6.442	4.500	+ 4.07

There is actually some error introduced in computing the maximum bending moments in this manner by assuming full live panel loads on all the points loaded. For instance, a load at Point 6 produces a negative moment at the crown, while a load at Point 8 produces a positive moment; but a full panel load could not be placed at Point 8 without having at least a half panel load at Point 6 which would reduce the positive moment produced by the load at Point 8. If an equivalent uniform load is used, it should actually be placed as shown in Fig. 71, and in this case the panel loads and moments for the maximum positive moment at the crown would be as given in Table 45.

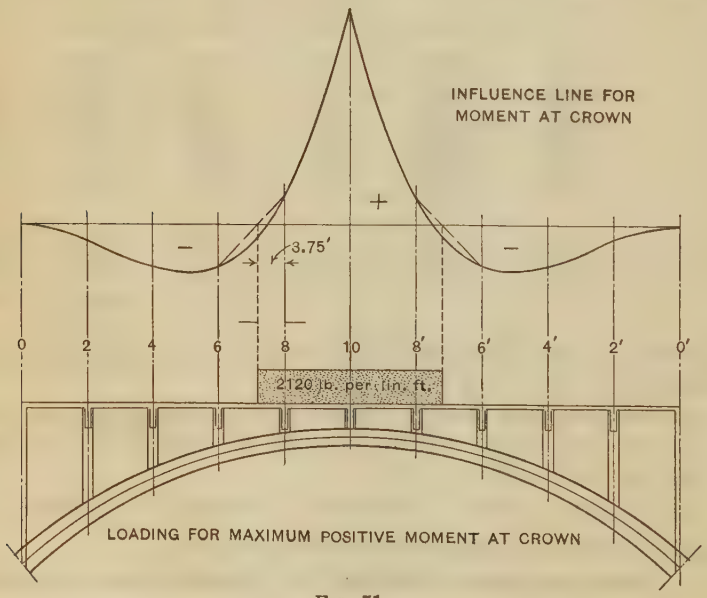

Fig. 71.

TABLE 45.—Positive Live Load Moment at the Crown.

Point loaded.	$\frac{M}{W}$.	W, in pounds.	M, in foot-pounds.
6	−0.76	1 600	− 1 220
8	+0.51	16 400	+ 8 360
10	+3.89	20 000	+77 800
8	+0.51	16 400	+ 8 360
6	−0.76	1 600	− 1 220
$\Sigma M =$	.....	.....	+92 080

This gives a moment of 92 080 ft-lb. instead of 98 200 ft-lb. as computed previously. This is, however, a refinement which may not be justified as the error is on the safe side. Also, in some cases of concentrated loading it might be possible to obtain a full load at one point with no load at the adjacent point.

For a live load distributed along the arch rib, Fig. 50 gives for the maximum positive live load moment at the crown:

$$M_c = 0.00453\ w\ l^2$$
$$= 0.00453 \times 2\,120 \times 93.3^2 = 83\,500 \text{ ft-lb.}$$

It is seen that in this case the concentration of loads at the columns has the effect of increasing the critical live load moment at the crown about 10 per cent. The effect on the moments at the springing line is much smaller. For instance, Fig. 53 gives for a distributed load:

$$M_s = -0.02135 \times 2\,120 \times 93.3^2 = -393\,000 \text{ ft-lb.}$$

and for concentrated loading from the influence lines:

$$M_s = -19.09 \times 20\,000 = -381\,800 \text{ ft-lb.}$$

In this case, the moment given for distributed loading is slightly greater because it includes the effect of the extra half panel load at the end, which is carried on the abutment column.

In making a preliminary design of an arch rib, an allowance must be made for the increase in the moments under the load points in an open spandrel design. The effect of the concentration will vary inversely with the number of panels, being slight when the number is large.

The dead load moments will next be computed for the crown and springing line sections. If all the dead loads were distributed, the dead load moments would be extremely small, as the arch axis and the pressure lines would practically coincide. Because of the concentration of loads at the panel points, there will be a positive bending under each column and a negative bending between the columns. Table 46 shows the results.

It should be noted that the ΣM computed from the influence lines is corrected because part of the load is not actually concentrated on the arch at the columns. The weight of the rib itself is distributed. For the crown, the correction can be found from the influence lines by taking the difference between

TABLE 46.—Dead Load Moments.

Point.	Dead load, in pounds.	Crown.		Springing Line.	
		$\frac{M}{W}$.	M, in foot-pounds.	$\frac{M}{W}$.	M, in foot-pounds.
2	58 800	−0.25	− 14 700	−6.67	−382 000
4	51 250	−0.74	− 37 950	−7.75	−397 000
6	46 800	−0.76	− 35 600	−4.67	−218 500
8	44 350	+0.51	+ 22 600	+0.39	+ 17 300
10	44 950	+3.89	+175 000	+4.90	+220 000
8′	44 350	+0.51	+ 22 600	+6.89	+305 000
6′	46 800	−0.76	− 35 600	+6.11	+286 000
4′	51 250	−0.74	− 37 950	+3.59	+184 000
2′	58 800	−0.25	− 14 700	+1.03	+ 60 500
$\Sigma M =$			+ 43 700		+ 75 300
Correction			− 13 300		− 31 400
Dead load, M			+ 30 400		+ 43 900
Waddell			+ 34 000		+ 43 000

the ordinate at the center panel point and the average ordinate for a distance on each side half way to the next panel point. This is:

$$3.89 - 2.80 = 1.09 \text{ ft-lb.}$$

The weight of the rib at the center panel is 12 200 lb. Hence the correction is:

$$-1.09 \times 12\,200 = -13\,300 \text{ ft-lb.}$$

At the springing line, a correction is also necessary to allow for the negative moment produced by the weight of the rib that extends from the springing line section half way to the next panel point. This moment could be found from the influence line, but is practically the same as if that piece of the rib were a simple cantilever extending out from the abutment. The weight is:

$$W = \frac{9.33}{2} \times 4.33 \times \frac{4.82 + 4.05}{2} \times 150 = 13\,460 \text{ lb.}$$

and the moment will be:

$$M = -\frac{9.33}{4} \times 13\,460 = -31\,400 \text{ ft-lb.}$$

The total moments and thrusts at the crown are summarized in Tables 47 and 48 and the negative moments at the springing line in Table 49.

In Table 49, the rib-shortening thrust must be computed by using the dead load, H, and the live load, H, produced by the loading that gives maximum negative moment at the springing line. This is:

$$\text{Dead load, } H = +299\,440$$
$$\text{Live load, } H = 1.383 \times 20\,000 = +27\,660$$
$$\text{Total } H = +327\,100 \text{ lb.}$$
$$H_{RS} = -0.0254 \times 327\,100 = -8\,310 \text{ lb.}$$

TABLE 47.—THRUST AT CROWN FOR MAXIMUM POSITIVE MOMENT, IN FOOT-POUNDS.

	From influence lines.	Waddell.
Dead load	+299 440	+308 000
Live load	+ 73 440	+ 73 000
Rib-shortening	— 9 470	— 9 000
Temperature	— 25 800	— 29 700
Total without temperature	+363 410	+372 000
Total with temperature	+337 610	+342 000

TABLE 48.—MAXIMUM POSITIVE MOMENT AT CROWN, IN FOOT-POUNDS.

	From influence lines.	Waddell.
Dead load	+ 30 400	+ 34 000
Live load	+ 98 200	+ 95 000
Rib-shortening	+ 37 600	+ 34 000
Temperature	+102 400	+114 000
Total without temperature	+166 200	+163 000
Total with temperature	+268 600	+277 000

TABLE 49.—MAXIMUM NEGATIVE MOMENT AT SPRINGING LINE, IN FOOT-POUNDS.

	From influence lines.	Waddell.
Dead load	+ 43 900	+ 43 000
Live load	—381 800	—372 000
Rib-shortening	—120 700	—129 000
Temperature	—375 000	—435 000
Total without temperature	—458 600	—458 000
Total with temperature	—833 600	—893 000

The excellent complete method developed by Mr. Robin for arch design is somewhat similar to that given in the paper. His formulas and tables for determining the curve of the axis, the variation of the thickness of the rib, and the bending moments and thrusts greatly simplify the work of design. The curve of the arch axis is a fourth degree parabola derived from the assumption that the load per linear foot of rib varies from the crown to the springing line as the ordinates of a parabola of the second degree. This method was discussed briefly on page 977. When the value of $\frac{w_s}{w_c}$ is as small as it was in the example just given ($g = 1.6$), the axis established by Mr. Robin's formula

is almost the same as that given by Equation (38). After the true pressure line has been determined, it would be possible to pass the axis curve given by Mr. Robin through the quarter-point of the pressure curve in order to get the curves to agree more closely. In this case, the value of $\frac{b}{a}$ would be established by the quarter-point ordinate instead of by the value of $\frac{Q}{6\,P}$.

Mr. Robin's formula for the variation of the rib thickness is of the same general form as that of the writer (page 1087), with $v = 2$. This will result in a rib slightly thinner in the haunch. It is interesting to note from Table 48 that reducing the thickness of the rib in the haunch had the effect of increasing the maximum bending at the crown when the temperature effect is not included and of decreasing the total combined bending, including the temperature effect. There may be no economy, therefore, in using a thinner rib than that given by Equation (49) unless the rise is low, so that the total combined bending, including the temperature effect, controls the design of the rib.

In many cases, Mr. Robin's formulas should give results as satisfactory as those of the writer. When the value of $\frac{w_s}{w_c}$ is not small, it appears that Equation (38) gives a closer approximation to the true pressure line than the assumption of parabolic variation of the load.

The value of $\int x^2\,d\,w$ mentioned by Professor Cross is obtained from Equation (54) and is:

$$\int x^2\,d\,w = \frac{l^3}{E\,I_c}\,\frac{1 + 3\,m}{48}$$

In spite of the serious limitations of the theory of elasticity as applied to concrete arches, the writer does not entirely approve of the pessimistic attitude of many engineers. Progress will not be made by discrediting what has proven to be the best theory available, but by experimental studies of its practical shortcomings so that it may be applied more intelligently. The fact that the more slender arches have been carefully designed by means of this theory may account for the lack of failures of structures of this type. The writer is pleased that the discussion has emphasized the dangers of the application of the theory without proper consideration of the practical aspects. Theory is sound only in so far as it is borne out in practice; it is to be hoped that the experiments now under way will add considerably to the engineer's knowledge of how the theory should be applied to the design of concrete arches.

The method which Professor Vawter suggests for the use of the diagrams and tables for checking existing arches should give good results. The analysis just made of the arch designed by Waddell will give an indication of the amount of error caused by a difference in the variation of rib thickness in the haunches. No doubt many arches have been built in the United States for which the value of m is less than 0.15, but the writer hopes that the proportion is not as great as Professor Vawter's figures would indicate.

An examination of the proportions of several hundred European arches has led the writer to believe that the use of ribs with $\frac{d_s}{d_c}$ much greater than 2.0 is limited to American practice and that it should be condemned for both economic and æsthetic reasons. Practically all the finest stone or concrete arches of Europe and the most beautiful ones in the United States have values of m greater than 0.15. In most cases the value of $\frac{d_s}{d_c}$ lies between 1.5 and 2.0, and the average value seems to be about 1.75.

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Paper No. 1569

DEVELOPMENT OF HIGHWAY TRAFFIC IN CALIFORNIA*

By L. I. Hewes,† M. Am. Soc. C. E.

With Discussion by Messrs. Arthur E. Loder, Watt L. Moreland, and T. E. Stanton, Jr.

Synopsis

This paper compares for 1920 and 1922 the highway traffic count of the California State highway system and adjoined county roads as taken by the United States Bureau of Public Roads and the State Highway Commission. For these two State-wide traffic counts there were 103 stations identical, covering 2 150 miles of the main State highway. The total average of all traffic per day for 1920 was found to be 1 382 vehicles and for 1922, 2 037 vehicles, an increase of 47.4 per cent. During this interval the motor-vehicle registration increased 54 per cent. In the 1922-23 count, 138 additional counting stations on State highways and 190 stations on adjoined county highways, making a total of 431 stations, were used. The paper gives an analysis of the character and volume of traffic at all these stations. All counts are based on an "equivalent" 16-hour day from 6:00 A. M. to 10:00 P. M. This day is built up by combinations of 8-hour fractions.

In California, truck traffic in 1920 was 125% of the total number and in 1922, 11.3 per cent. In the winter of 1922-23, truck traffic was about 8% of the total, corresponding with an average of about 10% for the whole year 1922-23. On the adjoined county roads, truck traffic was found to be about 11.5% of the yearly total.

By the use of "traffic profiles", which present the typical daily traffic for all the State highway routes, the total annual movement of all traffic on

* Presented at the meeting of the Highway Division, Pasadena, Calif., June 19, 1924.
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California State highways has been computed for 1920 and 1922. For the latter year, it was about 1 500 000 000 vehicle-miles.

A study was made of the characteristics of all truck traffic and some interesting figures are presented with respect to commodities hauled, overloads, length of haul, speed of operation, etc.

The number of persons per motor vehicle in California for 1923 was found to be 3.47, with an automobile registration of 1 100 283.

The supply of California motor vehicles is compared with the discharge or junking. From the curves of supply and discharge, it was calculated that the average life for the period beginning in 1915 was about 8 years.

The necessity of making better rates for gasoline and registration taxation is especially emphasized. The relationship between the public operating revenue on the highways and the necessary operating expenses for such highways is found by computing the total annual traffic on two main State routes totaling 857 miles, between Los Angeles and San Diego, in the south, and Sacramento and San Francisco, respectively. At the current taxation, per vehicle-mile, the actual traffic on the selected routes does not meet the operating requirements of the highway, which are found to be $2 428 per mile for maintenance, reconstruction, and improvement. The indicated operating income per vehicle-mile for 1923, based on the total traffic and the total gas and registration revenue, was about $0.0018. At this rate there was a deficit of $766 per mile based on the actual traffic and actual expenditures on the 857 miles observed. On the other hand, the actual returns to the State on the gas and registration revenue indicate a rate of about $0.005 per vehicle-mile for the actual operation when prorated to the State highways only.

The position of California with respect to highway transportation is unique. Its large motor-vehicle registration, outdoor climate, and well developed highway system combine to make traffic exceptional. The measurement of this traffic and its bearing on other highway statistics is of general interest and value to highway engineers and highway economists. An indefinite number of statistical relations from the State-wide traffic counts might be set up, but it is the object of this paper to present only the major features of the traffic development and their economic significance.

Traffic Count of 1920.—Before 1920 a State highway traffic count had never been taken. That year, in connection with a study of the California highway system by the United States Bureau of Public Roads, a preliminary traffic count was made, on the basis of one equivalent 16-hour day, at 103 stations, on 2 150 miles of the main State highways. The count was intended to sample the summer traffic (June 1 to November 1) only. It was taken in the interval, August 7 to October 14, and gave a daily average of 1 382 vehicles per station between 6:00 A. M. and 10:00 P. M. Of this number, 12.5% was motor trucks and 2.3% horse-drawn vehicles.

Beginning about 1915, there had been certain prior traffic counts made by Kern and Los Angeles Counties on selected highways, which showed a pro-

gressive increase in average daily truck traffic between 1915 and 1920 of 25 to 140 in Kern County, and of 95 to 330 in Los Angeles County.

The census of 1920 was intended principally to determine the usefulness of the California State highways and to measure the relation between traffic and existing conditions on 2 150 miles of roads. There were introduced, for conveniently determining the total traffic, traffic diagrams, or traffic profiles, on which the State highway routes were shown horizontally, and the daily traffic of motor trucks and vehicles vertically, both to suitable scales. (See Figs. 1 and 2.) The area below the traffic curves, in proper units, thus indicated the total daily traffic in vehicle-miles. From these diagrams for the several routes, the total daily use of the State highways was developed. Thus, the daily summer traffic in 1920 was estimated as approximately 2 500 000 vehicle-miles for the main surfaced highways, and the total for the summer interval alone, June 1 to November 1, for all the routes measured, as approximately 375 000 000 vehicle-miles.

In making this count it was necessary to observe the 8-hour law and instead of one continuous 16-hour day from 6:00 A. M. to 10:00 P. M., it was frequently found convenient to build up an equivalent day by two or more shifts. Hence, the count for the equivalent day represents the assembly and average of 8-hour fractions of from 2 to 12 calendar days at the various stations. Several 24-hour counts showed the 16-hour traffic to be 88% of the total for the 24 hours.

A questionnaire sent out to 21 000 truck owners brought 1 930 replies, in which the attempt was made to answer the questions of average distance operated daily, average weight, advantageous speed, and miles per gallon of gas. The questionnaire involved considerable labor and expense and, although it was not satisfactory, it had certain value as a check. It showed that the trucks of larger capacity (from 3½ to 7 tons) operated a greater average daily distance and at a less speed than the smaller ones; and that agricultural products constituted the main commodity hauled, with return hauls of groceries, provisions, and general merchandise, but pre-eminently of building materials and machinery. The questionnaire indicated that the average net load in 1920 was 2.1 tons, and that the inbound (return) tonnage had a ratio to the outbound of 1.38.

In 1920, attention was also given to the passenger bus lines, which had already reached an advanced stage of development. Bus lines are regulated by the State Railroad Commission, under the Act of May 1, 1917, as amended May 13, 1919.* Bus owners before operating must secure, after a hearing, a certificate of public convenience and necessity. The Commission regulates rates and conditions and requires reports on operations. Under its jurisdiction and protection against unfair and unnecessary competition and enforcement of schedules, etc., the passenger bus business has developed until, in 1920, there were 103 independent bus lines, excluding those with insufficient data. In the operation of these bus lines, were involved 3 075 miles of State highways and an average daily movement of 65 220 vehicle-miles, of which

* This law exempted bus lines operating prior to May 1, 1917.

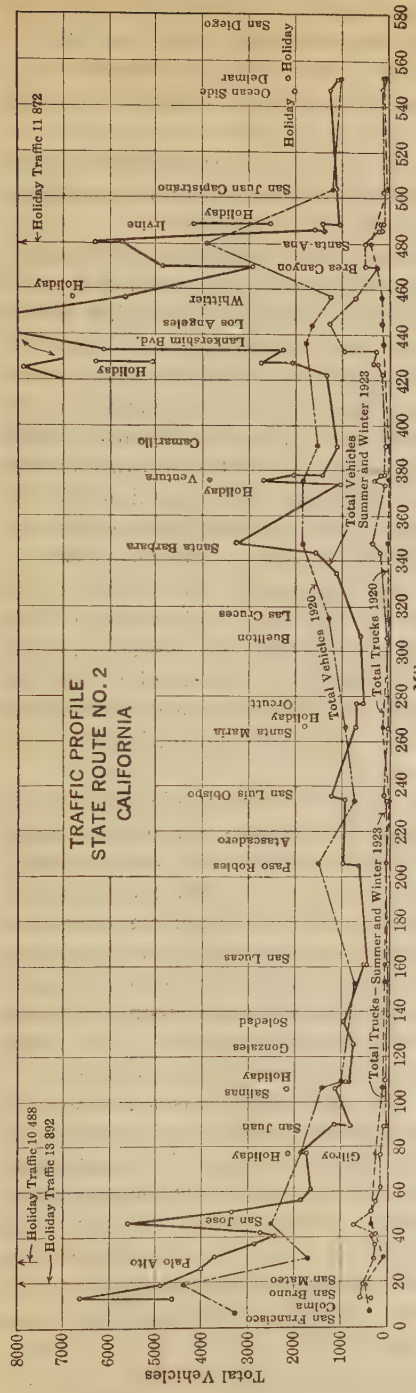

TRAFFIC PROFILE
STATE ROUTE NO. 2
CALIFORNIA
Holiday Traffic 10 488
Holiday Traffic 13 892
Holiday Traffic 11 872
Total Vehicles
Miles
San Francisco
Colma
San Bruno
San Mateo
Palo Alto
San Jose
Gilroy
Holiday
San Juan
Salinas
Holiday
Gonzales
Soledad
San Lucas
Paso Robles
Atascadero
San Luis Obispo
Santa Maria
Holiday
Orcutt
Buellton
Las Cruces
Santa Barbara
Holiday
Ventura
Camarillo
Holiday
Lankershim Blvd.
Los Angeles
Whittier
Holiday
Brea Canyon
Santa Ana
Irvine
Holiday
San Juan Capistrano
Holiday
Ocean Side
Delmar
San Diego
Total Trucks – Summer and Winter 1923
Total Vehicles 1920
Total Trucks 1920
Total Vehicles Summer and Winter 1923

FIG. 1.

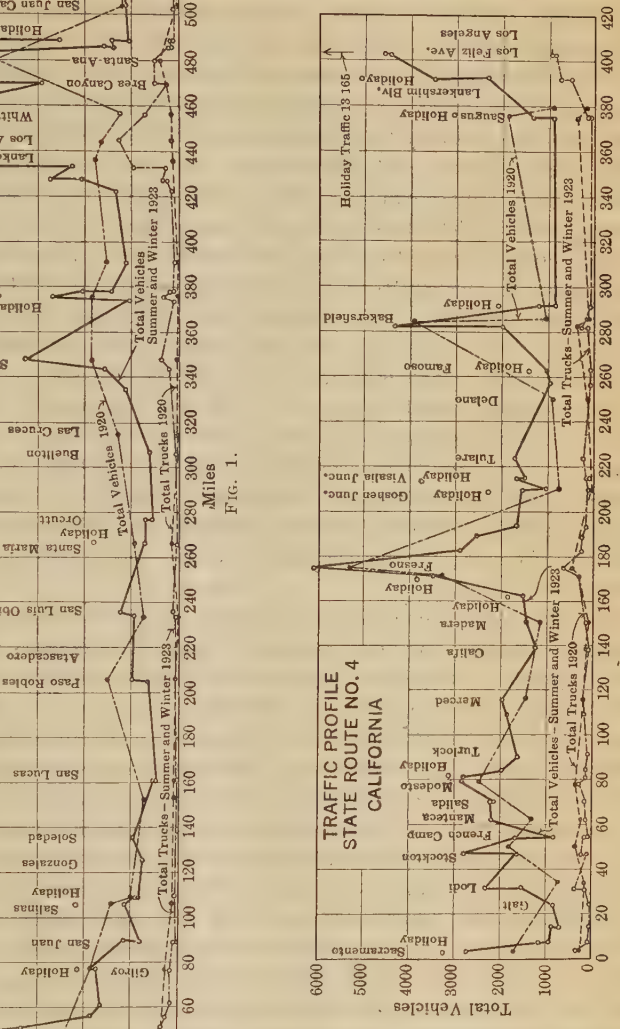

TRAFFIC PROFILE
STATE ROUTE NO. 4
CALIFORNIA
Holiday Traffic 13 165
Total Vehicles
Miles
Sacramento
Holiday
(Galt)
Lodi
Stockton
French Camp
Manteca
Salida
Modesto
Holiday
Turlock
Merced
Delhi
Madera
Holiday
Fresno
Holiday
Goshen Junc.
Holiday
Visalia Junc.
Tulare
Delano
Famoso
Holiday
Bakersfield
Holiday
Saugus
Holiday
Lankershim Blv.
Holiday
Los Feliz Ave.
Los Angeles
Total Vehicles – Summer and Winter 1923
Total Trucks 1920
Total Trucks – Summer and Winter 1923
Total Vehicles 1920

FIG. 2.

46 046 were on State highways only. The longest trip then possible by purchase of through transportation tickets over connecting lines was from El Centro, *via* San Diego, Los Angeles, and San Francisco, to Portland, Ore., a distance of 1 448 miles. The capacities of passenger busses of the larger companies ranged from 8 to 22.

Traffic Count of 1923.—At the request of the California State Highway Commission, the U. S. Bureau of Public Roads co-operated in a second State-wide traffic census in 1922,* the total cost of which was $50 000.

For comparison, it was essential to count traffic at the same 103 stations that were occupied in 1920. In addition, 138 stations were established, which completed the original count and extended its scope from 2 150 miles to 3 200 miles, this including graveled sections and some graded mileage in the State system. A simple form of traffic-record blank, comparable to that in 1920, was adopted. The classification of traffic was between passenger cars and trucks, again subdivided into light and heavy. Loaded and empty trucks were recorded separately. There was also provision for horse-drawn vehicles, busses, and trailers. Full instructions were issued giving the limits of the classes and details for filling in the headings and for tallying at cross-roads, etc. It was not found necessary to record the direction of traffic in 1922.

The summer traffic count for 1922 was commenced on August 27 and completed on September 20. This was followed by a winter count which began on January 25 and ended on March 3, 1923. The average summer traffic for all the stations in 1922 was found to be 2 037 total vehicles per day, an increase of 47.4% over the corresponding figure of 1920. For 1922-23 the daily average, during the winter alone, was 1 578, and during the whole year, 1 800.

In addition, 24 co-operating counties at 190 stations made traffic counts which, in general, included records of from 6 to 24 days per station, usually for 16 hours. The total number of stations at which traffic was counted in 1922-23 was, therefore, 431. On 135 of the county stations the counts synchronized sufficiently with the State counts to enable the resulting data to be included in State-wide statistics. The total daily arithmetic summer average of all vehicles on county roads was 722, except in Los Angeles County, where it amounted to 5 338. Since the county stations represent about 2 100 miles of county highways (estimated), adjoining the State highways, such county highways may be said to carry about 26% of the total traffic recorded.

Traffic profiles were constructed for the 1922 State census as shown on Figs. 1 and 2; the annual movement of motor-vehicle traffic on all State roads observed (about 3 200 miles) totaled 1 370 000 000 vehicle-miles. The daily average traffic on the same 2 150 miles counted in 1920 was, in 1922, approximately 2 750 000 vehicle-miles, an increase of about 13 per cent.† The 1922 count also represents an equivalent 16-hour day from 6:00 A. M. to 10:00 P. M. This 16-hour day, as in 1920, was built up by combinations and averages of about three thousand 8-hour fractions. For the most part, however, the 16-hour

* This census was actually completed in 1923.

† The apparent discrepancy in this percentage of increase and the 47.4% of increase in average traffic is due to the much greater accuracy of the 1922–23 traffic profiles.

days in 1922-23 were continuous and both the summer and winter average daily counts are representative of an average of approximately four days each.

During the 1922-23 traffic count, trucks were weighed at eighty stations where the traffic was sufficiently heavy to justify the expense. Approximately 13 000 trucks and 1 100 trailers were weighed. It was found that, numerically, winter trucking was to summer trucking about as 55 to 100, and that truck traffic was approximately 10% of all traffic. Trucks classed by inspection as heavy (of more than 2½ tons capacity) were about 38.3% of the total number; the weighing statistics indicated, however, that 45.1% were heavy. About 55% of the total trucks were found to be loaded, in counting, and about 68% in the weighing; when traffic was very dense empty trucks were not weighed. Nearly 60% of the loaded trucks were overloaded with reference to their rated capacities. Of the trucks weighed, 11.7% were overloaded with respect to the 1921 law (700 lb. per lin. in. width of tires), and of these overloads, 82%, overloaded on the front axle, and 60%, overloaded on the rear axle, could have been avoided by re-distribution of the load.

The average net load in 1922 was 3.3 tons. Trucks of capacities of ½ ton to 1½ tons constituted 24.5%, of 2 to 2½ tons, 31.8%, of 3 to 4 tons, 27.2%, and of 5 to 7½ tons, 16.5% of the total number. The trucks of large capacity (5 to 7½ tons), however, carried 25.3% of the net loads weighed. Approximately one-half the truck traffic from a given point worked within a range of about 21 miles, although the average haul was 31 miles.

In 1922, of motor freight and passenger bus lines of all kinds, 710 were franchised under the control of the Railroad Commission. Some of the passenger lines carried express also, and some carried freight. Bus operators report annually to the Railroad Commission. Class A automotive transportation corporations having an annual operating revenue exceeding $20 000, numbered 61, of which 17 exceeded $100 000.* There were approximately 600 lines with incomes of less than $20 000. Many of the smaller lines have United States mail contracts and carry passengers. The figures for 1922 show that the ratio of operating expense of the Class A lines to operating revenues was 93.3 per cent. The total net operating revenue was in excess of $500 000. The average length of bus lines is apparently 41 miles. There appears to be a tendency toward consolidation, the most successful lines acquiring the smaller ones. On some of the highways bus traffic has become a considerable fraction of the total traffic; for example, between San Francisco and San Mateo, there are 200 busses daily; between San Mateo and San José about 115; between Los Angeles and Whittier more than 200; and north of Santa Ana about 130. The average rate per passenger-mile is apparently about 5.4 cents. Of 246 bus lines classified as to the length of route, 80, representing the largest class, operated from 25 to 50 miles; the total length of these lines is approximately 10 500 miles.

In filing applications for certificates of convenience and necessity with the Railroad Commission, freight trucking lines must show the proposed rates, schedules, and freight classifications. These lines usually make their own

* Most of the lines having an operating income in excess of $100 000 were in operation prior to May 1, 1917.

classification, but a gradual standardization is being evolved by the larger operators. The present rates vary from 20 cents per ton-mile for short hauls to 8.3 cents per ton-mile for hauls of about 100 miles.

Comparison of 1920 and 1922 Traffic Counts.—In connection with the census of 1920, the U. S. Bureau of Public Roads attempted to ascertain certain characteristic or invariant elements connected with California highway transportation, such as the seasonal changes in traffic, the weekly periodic variation, the daily variation from hour to hour, the remainder of going and coming traffic, and the percentage of traffic between 6:00 A. M. and 10:00 P. M. Studies of speed were made for various classes of vehicles, with some useful results. All the 1920 results and deductions have since been refined and checked by the census of 1922.*

The seasonal variation (which was shown to exist in 1920 and merely approximated) was satisfactorily established in 1922, the ratio of total winter to total summer traffic being approximately 77%, except for trucks. Truck travel decreased much more than pleasure traffic during the winter, showing the importance of the harvest interval in California. The occurrence of only 58% of the weekly traffic during the first five days is characteristic. The remainder is on Saturday and Sunday. The daily variation of traffic is found to be characterized by a forenoon peak about 10 o'clock, and a greater afternoon peak at 5 o'clock. There is some deviation from this during the winter, when the afternoon peak seems to increase and the forenoon peak to decrease. The variation of total daily traffic differs somewhat from the truck traffic alone, for which the two peaks tend to be more nearly equalized and a greater percentage occurs prior to 6:00 A. M. The going and coming traffic so nearly equalizes (as it should) that, in 1922, no separate records of direction were made, and this simplified the counting. It was found in 1922-23 that about 92% of the total traffic occurred between 6:00 A. M. and 10:00 P. M., as compared with 88% in 1920. The higher percentage checks with the 1922 counts in several counties in Tennessee.

Speed tests on measured sections of the highway, made with a stop-watch in 1923, compared with the 1920 results without marked discrepancies. The 1922 tests were more accurate and show, on an average, a higher speed. Heavy passenger cars operate at high speeds and heavy trucks at the lowest speeds, but commercial passenger busses operate at the highest speeds of all, approximately 33.1 miles per hour on an average. Table 1 shows the average weekday traffic at the comparable 103 stations for the two years, 1920 and 1922.

On the 2 150 identical miles covered both in 1920 and 1922, motor traffic was found to constitute 98.8% of the whole, as against 97.7% in 1920. The increase in total daily traffic was 47.4%, or from a summer arithmetic average of 1 382 vehicles in 1920 to 2 037 in 1922. In the corresponding interval, the automobile registration had increased 54 per cent. On the county roads, also, 97.7% of all traffic in 1922 was found to be motor-driven.

* For details of the 1920 count see "Study of California Highway System", by U. S. Bureau of Public Roads, and for the 1922 count see "A Report of Traffic on State Highways and County Roads in California", 1922, both published by the California Highway Commission.

In 1922, special attention was given to motor trucks of which 13 000 were weighed at 80 stations. The ratio of winter to summer trucks counted was as 55 to 100.*

TABLE 1.—Average Week-Day Traffic at 103 Stations,

	Number of Vehicles.					Percentage of Total Vehicles.				
	Summer.		Winter, 1922.	Average per day, 1922.	Counties.	Summer.		Winter, 1922.	Average per day, 1922.	Counties.
	1920.	1922.				1920.	1922.			
Total vehicles.....	1 382	2 037	1 578	1 800	722	100.0	100.0	100.0	100.0	100.0
Total trucks only....	173	231	127	180	83	12.5	11.3	8.0	10.0	11.5
Passenger cars.....	1 146	1 724	1 388	1 547	604	83.0	84.7	88.0	85.9	83.7
Light trucks, loaded.	...	73	38	55	27	...	3.6	2.4	3.0	3.7
Light trucks, empty.	173	69	41	56	28	12.5	3.4	2.6	3.1	3.9
Heavy trucks, loaded		58	30	44	18		2.8	1.9	2.5	2.5
Heavy trucks, empty		31	18	25	10		1.5	1.1	1.4	1.4
Horse vehicles.....	32	24	17	21	17	2.3	1.2	1.1	1.2	2.3
Busses.....	31	35	31	33	6	2.2	1.7	2.0	1.8	.8
Truck trailers.....	....	23	15	19	12	....	1.1	0.9	1.1	1.7

By comparison with the results of the thorough traffic census taken in Connecticut in 1921 and 1922 by the U. S. Bureau of Public Roads, it is found that the truck traffic in California in general shows characteristics similar to that in Connecticut. In both States manufactured products predominate as the commodity hauled, and the percentage of empty trucks decreases with the increased length of haul. In California, light trucks (1½ to 2½ tons) were 56.3% of all the truck traffic and carried 40.1% of the total net tonnage; in Connecticut, similar light trucks were 66.6% of the total and carried 45.3% of all the commodities. There are, however, some interesting differences; for example, in California, there is a greater tendency for the loads to over-run the rated capacity of the truck; likewise the 1922 census showed that about one-half the trucks haul less than 21 miles, the average being approximately 31 miles, whereas, in Connecticut, the average truck haul was only 15.7 miles (for Connecticut trucks alone). In California, no particular correlation was observed between the width of the truck body and the overloading of the truck, although this correlation was observed in Connecticut.

Truck registration data in California in the past has not been satisfactory because, to 1924, commercial vehicles or trucks with pneumatic tires have been registered as automobiles and only vehicles with solid tires have been classed as trucks. This classification probably accounts for a decrease from 41 689 trucks registered in 1920 to 35 092 in 1921. Registrations for 1922 and 1923, however, show increases to 39 413 and 43 000 plus, respectively. In the 1920 census all trucks were counted as such irrespective of their tire equipment, and the daily average was found to be 173 at the 103 stations that

* See "A Report of Traffic on State Highways and County Roads in California", published by the California Highway Commission.

were comparable with the 1922 count. In 1922, the truck traffic was found to be 231 exclusive of the pneumatic tired vehicles, which, in harmony with the registration procedure, were classed as heavy automobiles. It is thought that the small increase shown in truck registration from year to year is probably due to the adoption of pneumatic tires to an increasing extent for all lighter trucks, particularly commercial vehicles of the delivery type, and also to the relatively large increase in this class of commercial vehicles operating in cities. The motor-vehicle law as amended in 1923 now provides for separate registration of commercial vehicles, irrespective of their tire equipment; in 1924, to April, there had already been registered approximately 106 000 pneumatic tired commercial vehicles, either trucks or passenger vehicles, and 35 000 solid tired trucks. The total (141 000) immediately reflects the change in classification from the preceding year.

Discussion.—The progress of motor-vehicle registration in California since 1907 is shown in Table 2. This increase of registration compares with that for the United States, as shown in Fig. 3, and with the increase of each of the five leading States, as shown in Fig. 4. Based on the 1920 census, with a conservative estimate of the increase of population, the number of persons per automobile in California in 1923 was approximately 3.47, and the average number of persons per automobile for the entire United States for 1923 was approximately 6.13. The ratio for California is not exceeded in any other State.

TABLE 2.—Motor-Vehicle Registration in California by Years.

Year.	Total Vehicles.		
	Number.	Increase.	
		Number.	Percentage.
1907*	10 020	.....	....
1907	14 051	4 031	40.2
1908	19 561	5 510	39.2
1909	28 633	9 072	46.4
1910	44 122	15 489	54.1
1911	60 779	16 657	37.8
1912	91 194	30 415	50.0
1913	100 000	8 806	9.7
1914	123 504	23 504	23.5
1915	163 797	40 293	32.6
1916	232 440	68 643	41.9
1917	306 916	74 476	32.0
1918	407 761	100 845	32.9
1919	477 450	69 689	17.1
1920	583 623	106 173	22.2
1921	680 614	96 991	16.6
1922	861 807	181 193	26.6
1923	1 100 283	238 476	27.7

* Prior to April 8, 1907.

It is impossible to calculate accurately the total annual operation of motor vehicles in California. An estimate of the total annual movement on all State and county highways from the 1922-23 traffic census, is 3 000 000 000 vehicle-miles. However, the indicated probable annual vehicular movement,

based on the consumption of gasoline for the last quarter of 1923, is about 8 500 000 000 vehicle-miles; this mileage* was computed from the factors of (1) a gasoline revenue (2 cents per gal.) for the last quarter of 1923, of

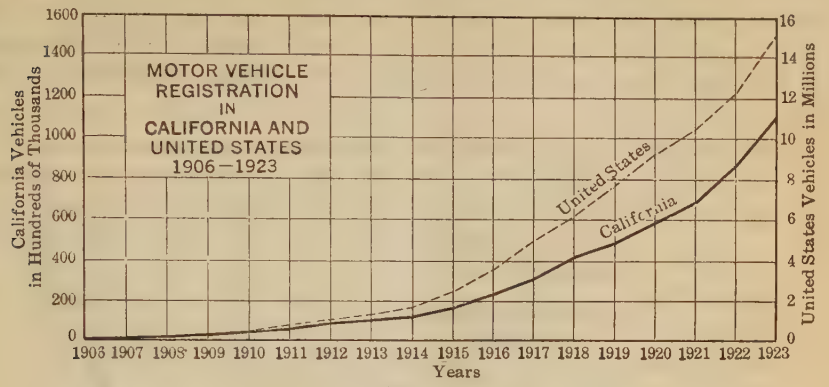


FIG. 3.

$2.29 per car; (2) an estimated movement of 15 miles per gal.; and (3) a registration of 1 100 000 vehicles. At once, a very significant fact appears, namely, that a vast movement of vehicles occurs in the cities and towns. For rate-making purposes it will be desirable in the near future to have traffic counts within the cities of the State.

The economic importance of the various elements of California highway traffic is evident. A major element is the motor-vehicle investment and the average life of the machine. Two curves are shown in Fig. 5 for the years, 1907-23, from which the average life of motor vehicles in California may be found up to about 1915. These two curves are denoted as "Total Supply", and "Total Discharge". The total supply curve for the years prior to 1921 is obtained by calculation. From the total yearly production of the United States there is allocated to California an annual number of new cars and trucks, fixed by the ratio of registration of California to the total registration of the country for the corresponding year. From 1921-23, inclusive, the actual number of new cars sold has been taken. The total discharge curve is then found by subtracting from the total supply the California registrations for corresponding years.† It is to be noted that both curves are cumulative; they represent the total machines supplied to or discharged from service at a given date. In Fig. 5 there are two interesting features: First, the vertical distance between the two curves represents approximately the registration for a given year; and, second, the horizontal distance between the supply curve and the discharge curve represents the approximate annual life of cars at the year corresponding to a point on the supply curve. It is seen that the indicated life of cars since 1915 was approximately 8 years.

* Allowing an increase for five months summer traffic and disregarding any industrial use of gasoline because of constantly increasing registrations.

† The dip of the discharge curve in 1918 is possibly due to demand for new cars exceeding production, which demand resulted in the return of discarded cars to service; or it may be that production figures for 1918 were incomplete.

As stated previously, the total traffic for 1923, outside of cities, was probably not less than 3 000 000 000 vehicle-miles. On the 3 200 miles of State highways the traffic count of 1923 shows a movement of 1 370 000 000 vehicle-miles. With the known traffic on approximately 2 100 miles of main county highways added, the figure is 1 850 000 000 miles, exclusive of Los Angeles County.* Adding as an estimate of Los Angeles traffic, 900 000 000 miles, and allowing for all extra traffic, 250 000 000 yields 3 000 000 000. With

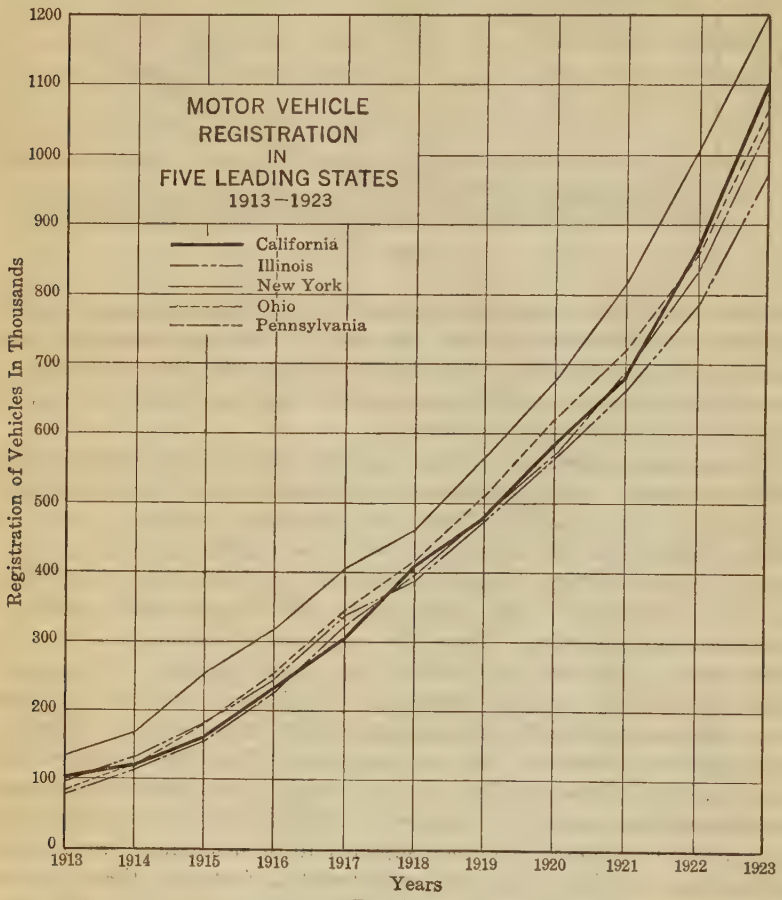
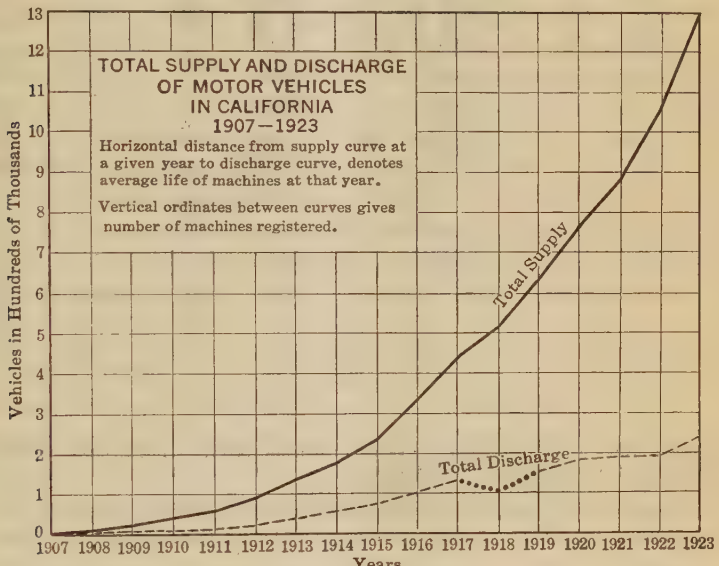


Fig. 4.

allowances for holiday traffic and unimproved sections of the total legal State system of 6 400 miles, the annual traffic on State highways alone for 1922-23 was probably not far from 1 500 000 000 vehicle-miles. Details of the State highway traffic studied by State routes are shown in Table 3.

* The Los Angeles County traffic is largely municipal in character. The daily average total vehicle movement was 5 338, and it is difficult to segregate the total county highway and State highway movement.

Relation of Highway Traffic to Motor-Vehicle Tax Rates.—A study of highway transportation will be necessary in connection with the taxation of traffic. There is a growing conviction that highway traffic must be sufficiently taxed to pay the cost to the public for maintenance and upkeep and perhaps for at least a part of the construction. Taxation of such form and for such purpose is essentially rate-making for highway transportation and should follow closely the principles of other rate-making for public utilities. The two principal forms of motor-vehicle taxation now established—the motor-vehicle license fee and the gasoline tax—have exhibited in the past rapid changes and readjustments. In the future, rate-making for highway traffic must have a more scientific basis than in the past; the present rates are only approximations based on intelligent guesses.


TOTAL SUPPLY AND DISCHARGE
OF MOTOR VEHICLES
IN CALIFORNIA
1907—1923
Horizontal distance from supply curve at a given year to discharge curve, denotes average life of machines at that year.
Vertical ordinates between curves gives number of machines registered.
Vehicles in Hundreds of Thousands
0 1 2 3 4 5 6 7 8 9 10 11 12 13
Total Supply
Total Discharge
1907 1908 1909 1910 1911 1912 1913 1914 1915 1916 1917 1918 1919 1920 1921 1922 1923
Years
FIG. 5.

With respect to the highway traffic income, the present is a period of rapid transition. Since 1905 the motor-vehicle license fees have progressively increased, and recently truck rates have been graduated on a capacity or weight basis. Rates have been established for passenger-carrying vehicles by classes, by weights, by capacity, and by horse-power. The present California law makes a basic rate of $3 for all vehicles, with a surcharge sliding scale for trucks by weight, the maximum of $40 being for a 10 000-lb. unladen truck. This surcharge appears to be about $25 for solid tired trucks. The gasoline tax in California is 2 cents per gal. The indicated gross revenue from gasoline in 1924 is at least $12 500 000. Prior to 1924 the motorists of California have paid a total of $34 939 338 to California State and county highways. There are now thirty-seven States with gasoline taxes, varying from

1 cent to 4 cents per gal. For 1923 the average motor-vehicle license fee for the entire country was $12.49; in California, it was $9.64.*

TABLE 3.—Average Vehicle-Miles per Day on State Routes.

Route.	On Comparable Sections of Highway.					Additional Sections of Highway Not Comparable with 1920.		
		1920.		1922.				
	Miles* of highway.	Number of stations.	Vehicle-miles, summer.	Number of stations.	Vehicle-miles, summer and winter.	Miles* of highway.	Number of stations.	Vehicle-miles, summer and winter, 1922.
1	300	10	218 200	20	206 000	96	5	20 000
2	554	21	787 300	50	994 600	...	..	.....
3	142	8	164 500	10	163 000	158	3	60 800
4	380	15	603 800	32	612 800	23	2	(67 000)†
5	125	11	165 600	23	166 600	...	..	.....
6	....	1	.....	..	.....	15	2	30 400
7	176	10	127 400	11	132 800	...	..	12 000
8	....	1	.....	..	.....	...	2	.....
9	70	4	57 000	10	87 600	...	..	24 600
10	....	1	36 000	..	.....	32	6	29 800
11	51	3	36 000	4	31 200	65	1	22 000
12	130	3	64 100	5	79 600	...	..	.....
13	.....	1	.....	..	.....	22	2	11 000
14	33	3	56 200	5	31 800	...	..	.....
17	....	1	.....	..	.....	26	5	6 000
18	....	..	.....	..	.....	40	3	5 400
19	30	2	49 600	5	44 600	...	..	.....
23	86	2	50 900	2	26 400	...	1	.....
26	122	2	56 700	5	74 600	51	3	49 000
27	....	..	.....	..	.....	40	3	15 600
28	....	..	.....	..	.....	176	2	41 000
29	....	..	.....	..	.....	102	2	19 800
31	....	..	.....	..	.....	80	2	23 800
37	....	..	.....	..	.....	115	2	34 800
Totals.....	2 149	101§	2 437 300	182	2 654 600	1 041	46‡	406 000

* Refers to miles of State highway covered in census, but does not necessarily equal total miles of highway composing the various routes.

† Los Angeles County road, not included in total.

‡ There were thirteen stations on routes which were not worked up as profiles.

§ Four stations not shown on profiles, or combined with other stations in 1922.

The question arises as to the relation that obtains between the financial requirements of the highways and the taxes on the highway traffic. Clearly, the public operating revenue on the highways and the necessary operating expenses of the highways themselves should be related. Therefore, there is presented herewith an analysis of the annual cost of operation and revenues of two combined important State highway routes in California which total about 857 miles.† These two routes are Nos. 4 and 2, running south from Sacramento and San Francisco, respectively, to Los Angeles and San Diego, with practically continuous hard surfacing, mostly concrete and concrete with asphaltic concrete top. The profiles of average daily traffic as derived by the 1920 and 1922 traffic counts are shown for these two routes, respectively, in Figs. 1 and 2. The costs of original construction on these routes are known,

* This figure does not include the gas tax effective October 1, 1923.

† Exclusive of mileage in towns and cities, which if included brings the gross mileage to about 975.

also the costs of reconstruction and improvement, and the approximate annual maintenance charge. From these the total annual cost per mile to the State for operating the combined routes is found to be approximately as follows:*

Bond requirements, 6% on $19 270 construction costs (exclusive of Federal Aid)	$1 156
Maintenance and reconstruction	1 327
Improvement	1 101
Total	$3 584

Of this total, the bond requirements are already met by general State taxes. There remains a balance of $2 428 to be derived from current revenue if this figure prevails for the succeeding years.

The actual portion of the total annual operating income from motor vehicles accruing to Routes 2 and 4 combined, totaling 857 miles, can first be determined by computing the revenue from the 2-cent gas tax and allotting a proper proportionate part† (9.22%) of the flat $3 fee and the surcharge license on trucks. From Table 2 the total vehicle-miles per day on the combined routes is 1 607 400. This figure must be increased for the night traffic and also for the week-end traffic. An increase of 8% for night traffic, and an addition of approximately 24% for week-end and special traffic, gives a total annual vehicle mileage for the two routes of approximately 784 000 000. There is then assumed an average movement per gallon of gas of 15 vehicle-miles with no allowance for the expense of collecting the gasoline tax. The average surcharge on the 43 000 trucks in the State in 1923 is assumed to be $25. Of these license fees 6% is chargeable to over-head. The following revenue results:

Total gas tax on 784 000 000 vehicle-miles at $0.001333		$1 045 072
9.22% of $3 license fees on 1 100 000 vehicles	$304 260	
9.22% of $25 surcharge fees on 43 000 solid tired trucks	99 115	
	$403 375	
Less 6% for collection expenses	24 202	379 173
Total revenues, Routes 2 and 4		$1 424 245
Average per mile		$1 662

* Construction, taken from the State records, includes general items amounting to 14% of the distributed items, and, up to June 30, 1923, had amounted to $17 616 088, from which is subtracted $1 101 566 of Federal Aid funds. The remainder from bond revenues, $16 514 522, is equivalent to $19 270 per mile.

Maintenance and reconstruction, $1 327 per mile, is the sum of an average of $823 of general maintenance and $504, prorated reconstruction, for the period from June 30, 1922, to June 30, 1923, which compares with corresponding averages for the preceding three years of $656 general maintenance and $536 prorated reconstruction—total, $1 192 per mile.

The prorated improvement figure of $1 101 per mile is the average per mile expended for the year June 30, 1922, to June 30, 1923.

Bond requirements of 6% include 1½% approximate amortization charges.

† The total annual vehicle movement is taken at 8 500 000 000 vehicle-miles on all roads and streets. The movement on Routes 2 and 4 is 784 000 000 vehicle-miles, or 9.22 per cent. Truck traffic is assumed as 10% of all traffic, but the prorated truck surcharge is $0.000126 per vehicle-mile and the license fee, $0.000388 per vehicle-mile.

This analysis of operating revenues and expenses is made from current data. Some adjustments, however, are required. As the truck registrations and total vehicle registrations for 1924 are not known, the 1924 rates are applied to the 1922-23 traffic figures. The total operating income per vehicle-mile on the 8 500 000 000 vehicle-miles annually, indicated for 1922-23, is $0.001847. This figure includes the prorated gasoline tax, $0.001333, the license fee, $0.000388, and the prorated truck surcharge, $0.000126. The indicated gross revenue is, therefore, about $15 700 000, which figure will doubtless be exceeded in 1924. Consequently, the apparent operating deficit of $766 per mile from the first analysis is subject to decrease unless the costs of maintenance, reconstruction, and improvement correspondingly increase with the traffic.

Another analysis may now be made. Since the State Highway Commission by law receives one-half the net motor revenues for maintenance and improvement of State highways, the current State highway income from this source is about $7 500 000. This income, divided by 1 500 000 000 vehicle-miles of indicated State highway operation, yields $0.005 per vehicle-mile of operating revenue. To the 784 000 000 vehicle-miles produced on Routes 2 and 4, there accrues then $3 920 000 ($4 574 per mile), or a surplus of $2 146 over the current requirements on this route. If the bond requirements were included in the charges, the surplus would reduce to $990 per mile. Routes 2 and 4 are the heaviest traveled long routes on the State highway system, and the operating revenue is greater in proportion to expense than the average of the entire system. A study of the entire system by the first method of analysis would reveal a larger proportionate deficit, and, similarly, by the second method probably a smaller surplus, than the figures here given for Routes 2 and 4.

It is interesting to examine the benefits from improved highways to the motor vehicles themselves. The cost of operating the motor vehicles in the United States has been variously analyzed, but the totals per mile do not differ greatly. Recent figures presented by F. R. White, M. Am. Soc. C. E., at a Congressional hearing show an indicated cost of about 10 cents per mile under average conditions for wearing surfaces of Portland cement concrete and asphalt or bituminous macadam.* This cost included depreciation, repairs, tires, oil and gasoline (stated at 85% of the total cost of operation), but not license fees, garage bills, insurance, etc., factors unaffected by the improvement of the highways. An average operating figure of about 10 cents per vehicle-mile may be checked by various computations and is probably representative of conditions in California for the average car. It is conservatively estimated by Mr. White that this figure represents a reduction of at least 2.5 cents over the comparable operating costs on unimproved highways.

The foregoing analysis necessarily contains some tentative assumptions, particularly with respect to the vehicle-miles per gallon of gasoline and the percentage of total annual operation of vehicles on the combined State highways and county roads. The analysis is given, however, to indicate a method by which highway operating costs and highway revenue may be compared. Manifestly, the difference between vehicle taxation and benefits to the vehicle

* *American Highways*, April, 1924.

by improved highways is still too wide to warrant over-refinement of the basic assumptions. Apparently, the benefit of the highway to the vehicle ($0.025) is at least five times the cost of upkeep of the highway to the vehicle only ($0.005), and this is probably the important element of modern highway construction.

There is perhaps no stronger reason for the continuation of State-wide traffic counts than the necessity for accurately measuring the annual traffic as a basis for determining the correct taxation of motor vehicles. The division of motor-vehicle revenues between State and local purposes requires further investigation. The principle, which was begun with the advent of State aid in Massachusetts and New Jersey in the early Nineties, of paying for country roads by city revenue, is being carried out in California in a new form under the present motor-vehicle taxation by reason of the evident preponderance of traffic in cities, from which the operating revenue accrues to the State and county highways. If the State highways are to be extended for the general benefit of the people of the State, this principle of taxing the city traffic for country roads must continue.

DISCUSSION

ARTHUR E. LODER,* ESQ.—The development of the highway traffic of California, as presented by Dr. Hewes, cannot fail to impress every one with the seriousness of the highway transport problem. He shows that within a period of two years the average summer travel on the State highway system has increased 47.4 per cent.

The combined registration of automobiles and trucks was 574 623 for 1920, in which year the first traffic census was taken, and 861 704 for 1922, during which the second census was taken. This increase of 50% in registration in the two-year period is slightly in excess of, and shows cause for, the increase of 47.4% in the average summer traffic found in the traffic survey. The increase in traffic shown during the two-year period under study has undoubtedly been exceeded by the two-year period, 1921 to 1923, during which motor-vehicle registration in California increased 61.6 per cent.

The average summer traffic may be expected to increase in approximately the same ratio as motor-vehicle registration. Both factors indicate the potential motor traffic. With generally improved roads the maximum traffic must be expected to come out on the highways in summer (and, in fact, on any occasion during the year) and to demand a reasonable degree of comfort and efficiency in the use of the highway.

It is the peak load of traffic that gives rise to most of the difficulty. This exceptional traffic comes on Sundays or holidays in answer to the demand for recreation. It is highly desirable that it be accommodated, in so far as practicable, but the matter of first concern, from an economic standpoint, is the giving of efficient highway service to the largest possible total volume of traffic, and this consideration will probably make it impossible to provide highway facilities of sufficient width and extent to handle all maximum traffic.

It is probable that some form of regulation may be found, which will have a tendency to smooth down the peaks in the traffic profile. In fact, present conditions are such that a large percentage of city motorists must remain at home on Sundays because of the limited highways. Although highway facilities appear to be good in the vicinity of centers of population, they are found to be actually very inadequate when compared with the large number of motor vehicles, which, in California, has reached the high mark of 1 to each 3.5 persons. These conditions, and the fact that city motorists want to do their driving in the country, indicate one of the principal reasons why city motorists are contributing financially to the major highway improvement in the country.

Although California has been the scene of an unusually rapid increase in the ownership and use of motor vehicles, this is a problem which is confronting all the States, and especially the densely populated centers, as shown by the fact that the registration of motor vehicles in the United States for 1923 showed an increase of 23.6% over the previous year.

There is no occasion for alarm or sensational attempts at guessing the answer. The fundamental laws of economics will regulate abnormal tendencies

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for the best ultimate good of the country and will finally make for harmony between the urge for motor transport and the ability of the public to pay the bill. In the final analysis, this will serve as a check on the development of the motor industry and of motor transport to prevent it from growing too far in excess of possible highway facilities or too far beyond the ability of man to supply himself with the remaining necessities of life. There is much comfort in the knowledge that, although man may closely pursue his winged imagination in its search for unlimited individual transportation, he must always remain sufficiently close to earth and to common sense to satisfy his instinctive demands for food and shelter, the preservation of the race, and the esteem of his fellow-man.

There is only a remote probability of reaching the so-called "saturation point" in ownership and operation of motor vehicles. The increase will continue much as in the past with occasional interruption or slowing down as financial periods swing from expansion to depression. The ability of the public to finance highways will likewise continue to increase but, as at present, it will always be found lagging several laps behind the demand. Obviously, the future will see such large funds going into the maintenance, reconstruction, and new building of highways, as to make present budgets appear as small beginnings.

If increase in the number of motor vehicles, combined with increase in population and highway facilities, continues as in the past few years, a registration of 2 500 000 motor vehicles may be expected in California by 1930. This would indicate a total traffic four times as great as that measured in 1922. To this will be added a material increase in traffic caused by motor guests. Approximately 28 000 cars from other States and countries toured in California during 1922 and more than 66 000 during 1923, or an increase of 135% in one year. If this condition existed when the interstate connections were still unimproved, what may be expected in the next few years in the way of motor guests, when a number of transcontinental highways have been improved to such an extent as to make the trip a pleasure instead of an adventure?

These considerations of highway traffic point to the necessity for immediate and continued activity on the completion of the State highway system and the widening or duplication of a number of the most important inter-city and suburban routes.

During the present year (1924), the California State Highway Committee of Nine is making a study of the status of the State highway system from an engineering and economic standpoint and is required to submit recommendations to the Governor and the next Legislature on such questions as State highway classification, desirable progress, and policies affecting order of construction, and a method of taxation which may equitably place the burden of cost and provide funds for continuing the work toward completion. These studies of the development of highway traffic point to the fact that the State will probably never find itself able to say that its highway system is completed.

The highways must continue to grow in length, width, and thickness, as long as traffic continues to increase as it has in the past.

Although it may be said that the State faces the question of providing means of raising approximately $200 000 000 for the completion of work which seems desirable on the State highway system, to provide for present traffic conditions, it is probable that even if this work is performed within the next ten years the State would then find itself facing an even larger financial problem to care for the volume of traffic which may then exist. The probable increase in traffic points to the desirability of now placing the State's highway program on a "pay-as-you-go" basis to prevent paying at the rate of $2.08 (principal and interest) for each dollar available for construction, as has been necessary—and justifiable—on the first $73 000 000 of bond money which has already gone into the work.

The seriousness of the highway traffic problem of the State, and the financial responsibility which it brings, suggests the desirability of considering ways and means by which the usefulness and efficiency of inter-city highways can be increased. Anything that will have a tendency to increase the number of vehicles which can be given efficient highway service over the road, will help toward solving the problem and will have a tendency ultimately to conserve funds. Some highways now handle a large total count of traffic on busy days, but when people are required to proceed so slowly and stop so frequently because of traffic jams at the frequent intersections and through each village, they soon lose all interest in the trip and take the first detour home or spend many aggravating hours on what should be a short trip. In such cases it is true that a large total number of vehicles pass over the road, but each driver will tell you that he receives only about 10% efficiency as far as highway service is concerned.

Highway service should be regarded as the number of vehicles passing over the road modified and reduced by the efficiency of service. Some day this factor will be given far more consideration than at present. This will ultimately bring about the location of inter-city highways in such manner as to pass by the side of intervening towns and villages instead of through their congested streets.

The speed of traffic and, therefore, the efficiency of highway service must be increased by reducing to a minimum the number of highway intersections and road crossings at grade and providing the smallest possible number of intersecting avenues leading into the adjacent towns or villages. The future inter-city highways will be located and built so as to be safe for continuous speeds of 35 to 40 miles per hour, and probably more, operating under such conditions that vehicles can follow each other more closely than would be safe on the present highways with their frequent intersections and interruptions of the stream of traffic. The future great highways between the largest centers of population will probably be laid out with a view to these considerations under new legislation yet to be devised, whereby the highways can be under the complete ownership and control of the State instead of being merely an easement over property which now exercises the right of creating as many

dangerous intersections and frontage obstructions as owners and land "butchers" can devise.

Under present conditions, an important suburban or inter-city highway is no sooner paved to an ample width than sub-divisions are laid out and store fronts erected. This is followed by an unlimited number of street intersections and continuous parking of vehicles. To make the situation worse, school officials, perhaps through misguided civic pride, frequently locate beautiful new school buildings directly on main traffic thoroughfares, thus subjecting school children to the greatest possible danger to life and limb. Primitive races may yet be found where youth is subjected systematically to all manner of physical hardship and danger with a view to making him agile in dodging dangers and useful in war, but in this civilization it is doubtful if there is any justification for such a system of training, and the more thoughtful communities will place their schools in quiet places.

The future express highways between large cities will probably consist of one or more high-speed motorways, fenced or walled on either side, except at intervals of perhaps 5 or 10 miles where turn-outs and inlets from intersecting avenues will be provided. The high-speed motorways will be located on land owned in fee by the State and safeguarded under future legislation so that it may perform its function of serving the greatest possible number of people in the most efficient manner. On both sides the high-speed roadway will be flanked with narrower but continuous local roads to which the abutting property will have free and unhindered access. At the limited entrances to the high-speed motorways grade separations will be located and wide turn-outs and inlets arranged so as to avoid the necessity for left-hand turns or crossings at grade on the express highway. Grade separations will be provided wherever transverse roads must cross. With such a layout local traffic will be fully protected from high-speed traffic and the high-speed traffic will not be interfered with by local and slow traffic. Local traffic will follow the parallel local roadways until reaching the first intersection where it may, if it desires, enter the high-speed motorway.

It is true that at present the foregoing layout would be necessary in only a few parts of the country, but it is believed the time will come when substantially such a plan will be in use. If motor transport is to continue its development along with inter-city through automobile traffic, there will come finally an irresistible demand for such a plan.

Watt L. Moreland,* Esq.—This excellent paper brings out a number of interesting facts. As a truck manufacturer, the speaker is particularly interested in the statements that 60% of the trucks are overloaded, that the average length of hauls is 31 miles (which is about twice the length of hauls in the Eastern States), that the number of persons per automobile in California is a little less than 3.5, and that the average speed of passenger busses is 33.1 miles per hour.

This overloading of trucks is a constantly increasing problem for the truck distributor due to the limiting of weights and capacities by State authorities

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and to the constantly increasing tendency to buy lighter trucks and overload them. While the figures show that although 60% of trucks are overloaded as judged by their rated capacities, only 11½% were overloaded according to tire capacities. Of course, highway engineers are not particularly interested in the capacity of motor trucks to take overloading and the consequent high maintenance costs and short lives. What particularly appeals to engineers is the overloading of the maintenance charges of highways. It is doubtful whether any practical way can be found to prevent overloading beyond the manufacturer's rating. There is such a difference in rated capacities according to the weight of the truck and the grades of steel, etc., entering into the construction, that it would seem to preclude any such limiting legislation.

The long mileage, the large number of cars, the high speeds maintained by busses in California are easily explained, of course, by the miles of magnificent scenery, the splendid highways passing through non-congested areas, and the rapid growth of suburban and country districts. The extension of these highways into the mountains and to the seashore makes it imperative that if people are to enjoy the beauties of Nature and to secure the full advantages of the highways, they must own automobiles. A citizen of California certainly lives a better life—a more healthful life—and gets a greater enjoyment because of the highways and passenger cars than citizens in other States who do not enjoy these advantages in the same degree.

The growth of Los Angeles, or Southern California, for that matter, has kept pace with the automobile and one of the reasons for this growth has been the transportation facilities afforded by the motor vehicle.

The absence of tenement houses in Los Angeles can be credited to the passenger car, which makes it possible to live in the suburban districts and still do business within the city. Nearly all the working people now have light cars in which they travel to and from work.

The great area of the State, with its comparatively limited railroad mileage, has offered, and does offer, a fertile field for the development of motor-vehicle transportation lines, both freight and passenger. The railroads are now coming to realize the advantages to them of motor vehicles as feeders; in fact, railroads all over the country are eliminating short-line roads and turning over this work to the motor vehicle.

One great advantage that comes to the owner of an automobile has been the lower maintenance cost due to the splendid hard-surfaced highways. Greater efficiency in transportation means a reduction in cost of living. Highways should not become the football of politics, but should be managed for the benefit of all the people.

Although the increase of traffic in the past has been phenomenal, it will be much greater in the future; undoubtedly, the bulk of all future transportation will be by motor vehicles. With the practical elimination of the local steam train, the electric lines, in both city and suburban use, are now dividing the traffic with motor trucks. The speaker expects to see electric lines and railroad lines utilizing State highway systems and motor-vehicle transportation to the utmost.

The State highway system must be enlarged. California was one of the first, if not the first, State in the Union to build highways; a great deal had to be learned. These first highways were not properly constructed for the traffic, but since that time there has been a constant improvement. As the traffic increases the width must be increased and a careful study of traffic made to determine the advisability of one-way main arteries as compared with extra wide highways; the laws will have to be modified perhaps to permit of passing heavy loaded vehicles on the right side instead of only on the left. A great deal of attention should be given to the movement of traffic on the roads, and ways and means should be devised for insuring a steady flow. The benefit from good highways is shared by every citizen.

It would seem that maintenance costs have been excessive. This was brought about largely by inadequate highways in the first place—the surface and thickness were not of the correct dimensions and not enough attention was paid to limiting the loads carried. Imperfect highways, overloaded—two features that are now largely corrected—undoubtedly are responsible for this high maintenance cost; on the newer highways, with the regulation of loads, this cost should rapidly decrease.

Because of the many factors involved it is difficult to determine the economical load. It would seem that the present weight limit, fixed by the capacity of the highways to carry the load, is proper, and that as the highways are strengthened and special road-beds built, this limit should be increased.

Motor vehicle manufacturers are endeavoring to do their part by better construction of their products, using better material, better distribution of metal, and better distribution of load. Two or three manufacturers are announcing six-wheel vehicles, a feature which enables them to carry added loads under the law. As time goes on, this will increase until the economic capacity of different vehicles is definitely determined.

One of the greatest needs is the elimination of obstructions, such as grades, hills, narrow roads and crossings, and roads through a valley which acts like a bottle neck. All these should be removed and traffic allowed to flow smoothly.

The immense areas that will be brought into cultivation due to the many Western reclamation projects, will undoubtedly require more highways; the development of the back countries and the agricultural districts means more highways. History teaches that the better the transportation facilities, the higher the civilization.

T. E. Stanton, Jr.,* M. Am. Soc. C. E.—The development of motor-vehicle traffic in the United States opens an interesting and broad field for study to the highway economist.

Californians have been prone to consider, as Dr. Hewes shows, that the position of California with respect to highway transportation is unique in that "the very large motor vehicle registration, the outdoor climate, and the well developed highway system of the State combine to make highway traffic exceptional." It is interesting to note, however, the extent to which the

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development in California during the last ten years has been paralleled by other States.

Motor-Vehicle Registration.—It is true, as published statistics show, that there are more motor vehicles in California in proportion to the population than in any other State. A further study of these statistics, however, apparently demonstrates that it is only in this one respect that California occupies a unique position or one at variance with the average conditions found throughout the Union.

Two very interesting tables have been published by the U. S. Bureau of Public Roads.* An analysis of the figures in these tables—one a summary of combined passenger car and motor truck registrations for the years 1913 to 1923, and the other the gross receipts from gasoline taxes during 1923 in the States levying such a tax—shows that California, instead of occupying a position either at one extreme or the other, lies remarkably close to the average development throughout the country during the last decade.

The development of highway traffic in California during this decade has been remarkable, but so also has been the development throughout the country as a whole. Whether this development has resulted from improved highways, or whether the improvement in methods of transportation has been the occasion of the demand for more and better highways, may be a debatable question, but there is evidence that these two impelling motives have gone hand in hand, each spurring the other to greater efforts.

Referring again to the statistics mentioned, it appears that in 1913 the combined passenger car and motor truck registration in all the States amounted to 1 258 062 and that ten years later, in 1923, the registration was 15 092 177, an increase of 1 100 per cent. In the same period, the registrations in California increased from 100 000 to 1 100 283, or an even 1 000 per cent.

The smallest percentage of increase was in North Dakota, with 620%, and the greatest in Oklahoma, with 10 120%, but even with this large percentage increase Oklahoma had a total registration in 1923 of only 307 000, as compared with 1 100 283 in California. It will thus be noted that, as far as percentage is concerned, the increase in California does not present any abnormal elements that do not apply with equal force throughout the country as a whole. Rather is the fact emphasized that the development of highway traffic has been and is a major problem confronting the whole country, a problem which offers the same difficulty of solution in other States as in California.

In the light of these statements, the two traffic census studies conducted by the U. S. Bureau of Public Roads in California in 1920, 1922, and 1923, as well as those being carried on by the California State Highway organization, afford statistical data of direct interest and value to other States and thus doubly justify the expenditure of Government funds for the purpose. In Table 4 will be found figures showing the percentage increase in each State.

* *Public Roads*, April, 1924, pp. 16–17.

The relatively small number of motor vehicles per square mile in California, however, does offer greater problems in financing than in the more populous States of smaller area where the bulk of the traffic can be served with less mileage of main arterial highways.

TABLE 4.—Showing Percentage Increase in Motor-Vehicle Registration—All States—During Ten Years from 1913 to 1923.*

State.	Pleasure Car and Truck Registration.			Percentage of Increase.		
	1913.	1918.	1923.	1913-18.	1918-23.	1913-23.
Alabama	5 300	46 171	126 642	771	175	2 290
Arizona	3 613	23 905	49 175	562	106	1 261
Arkansas	3 583	41 458	113 300	1 058	173	3 060
California	100 000	407 761	1 100 283	308	170	1 000
Colorado	13 000	83 244	188 956	540	127	1 351
Connecticut	23 200	86 067	181 748	271	111	684
Delaware	2 440	12 955	29 977	431	131	1 129
Dist. of Columbia	4 000	30 490	74 811	663	145	1 770
Florida	3 000	54 186	151 990	1 706	181	4 966
Georgia	20 000	104 676	173 889	423	66	769
Idaho	2 113	32 289	62 379	1 427	93	2 852
Illinois	94 656	389 620	969 331	311	149	924
Indiana	45 000	227 160	583 342	405	157	1 196
Iowa	70 299	278 313	571 061	296	105	712
Kansas	34 550	189 163	375 594	448	99	987
Kentucky	7 210	65 884	198 377	814	201	2 651
Louisiana	10 000	40 000	136 622	300	242	1 266
Maine	11 022	44 572	108 609	304	144	885
Maryland	14 217	74 666	169 351	425	127	1 090
Massachusetts	62 660	198 497	481 150	209	149	667
Michigan	54 366	262 125	730 658	382	178	1 244
Minnesota	46 000	204 458	448 187	344	119	875
Mississippi	3 850	48 400	104 286	1 157	115	2 608
Missouri	38 140	188 040	476 598	393	153	1 152
Montana	5 916	51 053	73 828	762	45	1 148
Nebraska	13 411	173 374	286 053	1 191	65	2 030
Nevada	1 091	8 159	15 699	648	92	1 340
New Hampshire	8 237	24 817	59 604	202	140	623
New Jersey	51 360	155 519	430 958	203	177	739
New Mexico	1 898	17 647	32 032	830	81	1 586
New York	134 495	459 288	1 204 213	242	162	795
North Carolina	10 000	72 313	246 812	623	241	2 368
North Dakota	15 187	71 678	109 266	371	52	620
Ohio	86 156	412 775	1 069 100	379	159	1 140
Oklahoma	3 000	121 500	307 000	3 950	153	10 133
Oregon	13 975	63 324	165 962	353	162	1 088
Pennsylvania	80 178	394 186	1 043 770	391	165	1 202
Rhode Island	10 295	36 218	76 312	252	111	642
South Carolina	10 000	55 492	127 467	455	129	1 175
South Dakota	14 457	90 521	131 700	526	45	811
Tennessee	10 000	63 000	173 365	530	175	1 634
Texas	32 000	251 118	688 233	685	174	2 050
Utah	4 000	32 273	59 525	707	84	1 390
Vermont	5 913	22 553	52 776	281	135	790
Virginia	9 022	72 228	218 896	701	203	2 326
Washington	24 178	117 278	258 264	386	120	970
West Virginia	5 144	38 750	157 924	654	308	2 970
Wisconsin	34 346	196 253	457 271	472	133	1 233
Wyoming	1 584	16 200	39 931	924	146	2 415
Total	1 258 062	6 146 617	15 092 177	389	146	1 100

* Based on Table II, p. 16, Vol. 5. No. 2, *Public Roads*, April, 1924.

Gasoline Tax.—A study of the revenue per motor vehicle derived from the gasoline tax offers another interesting line of investigation. The 1923 session of the California Legislature passed a Gasoline Tax Bill calling for a tax of 2 cents per gal. The tax went into effect on September 30, 1923, so

that the income for only one-quarter of the year is available for purposes of comparison.

In estimating the probable revenue which would accrue from this method of taxation it has been customary to assume that because of the mildness of the climate in the populous sections of California the year-round traffic would average somewhat higher than in States with a more rigorous winter climate and that the corresponding revenue per car for each cent of gas tax would be somewhat greater.

As will be seen from Table 5, such an assumption was not well founded. It is true that the revenue for only one-quarter of the year, in the late fall and early winter, is available for direct comparison; however, the receipts for the first quarter of 1924 have been little in excess of the preceding quarter, thus indicating that, although the spring and summer months will undoubtedly show an increase, the average for the year will not be materially in excess of that shown in Table 5 for the last quarter of 1923.

For purposes of comparison it has been necessary to determine the probable revenue in each State in one year for each cent of tax. This has been difficult owing to the fact that the tax was in effect in a majority of the States only part of the year or that a change in rate was put into effect during the year.

The average revenue per car per State is found to be $4.46 for each 1 cent of tax. The corresponding revenue per car in California for the last quarter of 1923 was $4.58.

Several States, notably Texas with a tax of $3.26 per car, show an unexplainedly low return, whereas other States with a rigorous winter climate give remarkably high returns, notably New Hampshire, the highest of all, with $6.56 per car, and Maine with $5.45 per car. The four Southern States of Alabama, Florida, Georgia, and Louisiana, show a high average return of $5.41 per car. Whether it is the poor condition of the highways in these States resulting in a less mileage per gallon of gasoline or some other unsuggested cause or causes is for the investigator of the future to determine. Without doubt the variation in revenue per car in the different States offers a decidedly interesting field for study.

Construction and Maintenance Problems.—The rapid development of traffic in California presents many serious problems, not the least of which are those of design and financing. These two problems are, in fact, the major ones and to a very great extent must be considered together.

When State highway construction was started in California in 1912, there were approximately 91 000 registered motor vehicles; a year later, there were only a few more than 100 000.

A State highway system contemplating the construction of more than 3 000 miles of roads had been laid out and $18 000 000 (or an average of less than $6 000 per mile) had been provided for the construction of this system.

The Highway Commission and its Chief Engineer undertook to accomplish as much as possible with this small allotment. Having in mind the relatively limited traffic of that day, it adopted as a minimum pavement standard an unsurfaced Portland cement concrete base, 15 ft. wide and 4 in.

TABLE 5.—Gasoline Taxes, 1923.*

State.	Period, 1923.				Period, 1923.				Rate, January 1, 1924, in cents.	Gross receipts, 1923.	Estimated receipts in one year for each cent gas tax, based on 1923 gross receipts.	Registration, 1923.	Revenue per car.
	From	To	Months.	Rate, in cents.	From	To	Months.	Rate, in cents.					
Alabama	.....	.....	.....	....	Mar. 1	Dec. 31	10	2	2	$1 133 085.49	$679 851.29	126 642	$5.37
Arizona	Jan. 1	June 9	5.3	1	June 9	Dec. 31	6.7	3	3	474 123.04	224 000.00	49 175	4.35
Arkansas	Jan. 1	Apr. 1	3	1	Apr. 1	Dec. 31	9	3	4	1 219 198.75	487 679.50	113 300	4.30
California	.....	.....	0	0	Oct. 1	Dec. 31	3	2	2	2 518 893.00	5 037 786.00	1 100 283	4.58
Colorado	Jan. 1	Aug. 1	7	1	Aug. 1	Dec. 31	5	2	2	846 353.12	597 425.73	188 956	3.16
Connecticut	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	880 222.70	880 222.70	181 748	4.84
Delaware	.....	.....	.....	....	Apr. 22	Dec. 31	8.3	1	2	88 579.28	128 066.43	29 977	4.27
Florida	Jan. 1	July 1	6	1	July 1	Dec. 31	6	3	3	1 641 042.25	820 521.12	151 990	5.40
Georgia	Jan. 1	Oct. 1	9	1	Oct. 1	Dec. 31	3	3	3	1 502 503.49	1 001 668.99	173 889	5.76
Idaho	.....	.....	.....	....	Apr. 1	Dec. 31	9	2	2	396 487.18	264 324.78	62 379	4.24
Indiana	.....	.....	.....	....	June 1	Dec. 31	7	2	2	2 906 428.25	2 491 224.21	583 342	4.27
Kentucky	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	680 435.30	680 435.30	198 377	3.43
Louisiana	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	754 437.85	754 437.85	136 622	5.52
Maine	.....	.....	.....	....	July 7	Dec. 31	5.8	1	1	286 076.97	591 883.39	108 609	5.45
Maryland	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	2	688 304.02	688 304.02	169 351	4.06
Mississippi	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	467 855.53	467 855.53	104 286	4.49
Montana	Jan. 1	July 1	6	1	July 1	Dec. 31	6	2	2	441 249.10	294 166.06	73 828	3.98
Nevada	.....	.....	.....	....	Mar. 20	Dec. 31	9.3	2	2	115 843.24	74 737.57	15 699	4.76
New Hampshire	.....	.....	.....	....	July 1	Dec. 1	5	1	2	163 064.64	391 375.13	59 604	6.56
New Mexico	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	165 000.00	32 032	32 032	5.15
North Carolina	Jan. 1	Apr. 1	3	1	Apr. 1	Dec. 31	9	3	3	2 909 904.74	1 163 961.90	246 812	4.72
North Dakota	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	1	461 081.71	461 081.71	109 266	4.22
Oklahoma	.....	.....	.....	....	July 1	Dec. 31	6	1	1	599 000.00	1 198 000.00	307 000	3.90
Oregon	Jan. 1	May 24	4.8	2	May 24	Dec. 31	7.2	3	3	1 958 141.37	753 131.30	165 962	4.52
Pennsylvania	Jan. 1	July 1	6	1	July 1	Dec. 31	6	2	2	5 491 522.66	3 661 015.10	1 043 770	3.52
South Carolina	Jan. 1	Mar. 23	2.8	2	Mar. 23	Dec. 31	9.2	3	3	1 511 452.56	546 308.16	127 467	4.28
South Dakota	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	2	624 692.44	624 692.44	131 700	4.74
Tennessee	.....	.....	.....	....	Apr. 1	Dec. 31	9	2	2	812 356.68	541 571.12	173 365	3.12
Texas	.....	.....	.....	....	June 15	Dec. 31	6.5	1	1	1 215 623.36	2 244 227.74	688 233	3.26
Utah	.....	.....	.....	....	Mar. 8	Dec. 31	9.7	2½	2½	404 085.81	200 000.00	59 525	3.36
Vermont	.....	.....	.....	....	Apr. 1	Dec. 31	9	1	1	168 172.81	224 230.41	52 776	4.25
Virginia	.....	.....	.....	....	June 27	Dec. 31	6	3	3	1 556 920.99	1 037 947.32	218 896	4.74
Washington	.....	.....	.....	....	Jan. 1	Dec. 31	12	1	2	1 225 149.66	1 225 149.66	258 264	4.74
West Virginia	.....	.....	.....	....	July 27	Nov. 1	3.1	2	2	366 490.00	709 337.10	157 924	4.50
Wyoming	.....	.....	.....	....	Mar. 1	Dec. 31	10	1	1	140 161.62	168 193.94	39 831	4.22
Total	.....	.....	.....	....	.....	.....	.....	.....	.....	$36 813 939.61	.....	7 440 884	.....
Average	.....	.....	.....	....	.....	.....	.....	.....	.....	.....	.....	.....	$4.46

*Analysis based on Table IV, p. 17, *Good Roads*, April, 1924.

thick, costing on an average from $6 000 to $8 000 per mile, exclusive of the grading. To-day, pavements on main routes are being constructed not less than 20 ft. wide and 6 in. thick, increasing to 9 in. at the edge, at a cost averaging upward of $25 000 per mile, exclusive of the grading; and yet it can hardly be said with justice that the highway builders of ten years ago should have had sufficient vision to provide in highway construction for the needs of to-day, any more than engineers may be justified at present in looking ten years into the future and building accordingly, especially in view of the fact that insufficient funds have been provided to care for the needs of to-day, let alone those of to-morrow.

The question may well be asked, "Are we justified in looking even five years into the future and in planning accordingly?" The speaker believes so. Referring again to Table 4, the average increase in traffic throughout the United States for the five years, from 1913 to 1918, amounted to 388% and for a similar period, from 1918 to 1923, 146%, while in California during the same periods it was 308% and 170%, respectively. Thus, the rate of increase in California during the last five years was only a little more than one-half that for the preceding five years. It is very doubtful, however, whether the increase during the next five years will amount to even 100 per cent.

With the natural and normal increase which can be expected, and a reasonable increase in the 1924 registration and weight fee, combined with the 2-cent gasoline tax, the financing of the construction, reconstruction, and maintenance of the California State Highway System should not offer any serious problems.

Dr. Hewes points out the rather high annual maintenance, reconstruction, and improvement costs on State Highway Routes 2 and 4 since the original construction. These costs, however, can hardly be interpreted as indicative of the future. Many of the earlier 4-in. pavements were constructed on poor foundation soils. The combination of the poor soil, the narrow (15-ft.) width, and the exceedingly rapid passenger and truck motor development, put such a severe strain on this light pavement slab as to result in many cases in a rapid deterioration after a few years of service. Some of these sections have been completely resurfaced with from 4 to 5 in. of Portland cement concrete and others with from 1½ to 3 in. of asphaltic concrete, making a total thickness of from 5½ to 8 or 9 in.

At the same time, the width of pavement was increased to 20 ft. in most cases and the old base was thoroughly repaired before resurfacing. The resulting structure is undoubtedly a great improvement on the original construction and is much better able to stand up under modern traffic without undue and rapid deterioration.

Lack of funds has prevented the widening and thickening of much of the system, but many miles of the old 15 ft. by 4-in. bases, where the foundation conditions are good, are still giving excellent service and will undoubtedly continue to do so for some time to come if funds can be provided for the construction of a hard surface shoulder so that traffic can be diverted from its present concentration on the narrower width.

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Paper No. 1570

THE INFLUENCE OF THE AUTOMOBILE ON REGIONAL TRANSPORTATION PLANNING*

By George A. Damon,† Esq.

With Discussion by Messrs. Stephen Child and William T. Lyle.

The relation of the automobile to the transportation arrangement within Metropolitan Districts is one which should be discussed by all classes of technical men. It has been said that the difference between the civil engineer and the mechanical engineer is that the civil engineer designs structures to stand still, while the mechanical engineer builds machines to move. Transit facilities need both kinds of talent to locate and construct the permanent way and to furnish the equipment of motive power and moving vehicles. All classes of engineers are interested in the fundamental principles which underlie the most economical distribution of transportation units to secure the safest and most adequate systems for the movement of both passengers and freight.

Regional Planning.—The art of city planning is rapidly expanding to include the design of great Metropolitan Districts under the title of "Regional Planning." Conferences and commissions are discussing the "interdependence" in matters of common concern of the contiguous communities comprising the areas surrounding great cities. Of all these common welfare problems the question of intercommunication by means of an adequate, coordinated, and efficient system of transportation, is one of the most vital. The development and rapidly increasing use of the automobile now makes available an entirely new solution of the problem of securing the most desirable living conditions for human beings. The traffic congestion on highways and on city streets, due to the automobile, points the way to a new conception of city planning.

* Presented at the meeting of the City Planning Division, Pasadena, Calif., June 19, 1924.
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Expansion and Contraction.—All engineers know that every action has its corresponding and equal reaction. Thus, in regional development, the conditions at any time are the resultant of the forces of contraction (or congestion) and those of expansion (or decentralization). These opposing forces must be equal in order to maintain an equilibrium or a stable balance. Here, then, is the cue for design. Cities are bound to crystallize about centers where there will be great traffic congestion and hence they must be planned so as to make the most of the urban disadvantages.

To offset this congestion at the metropolitan center, outlying districts must provide for the freest circulation of traffic so as to secure for at least 50% of the population the benefits of suburban or even country surroundings. In working out the balance between city centers, satellite sub-centers, and open country, the automobile proves to be the vehicle of expansion while all other transportation units are really most efficient when working under conditions of comparatively higher density of traffic.

Congestion Regulated but not Cured.—Congestion at the centers is inherent in city development and "down town" streets become crowded almost to stagnation with automobile traffic. To widen these streets or open new ones, as they should and will be, is only to invite more automobiles, more traffic, and more congestion.

Building subways takes the cars off the street, and brings more automobiles in the resulting street space. Even the proposed "improvement" of upper decks for pedestrians, will only be temporary and there will soon be more congestion. Every positive solution which involves a physical improvement results in greater congestion, or, in other words, "the cure is worse than the disease." However, engineers will continue to effect these "cures" as fast as economic pressure produces the urge and the where-with-all, but they cannot hope ever to do away with congestion at city centers by any known means of actual physical betterment. As fast as voids are produced at the centers, the forces of centralization will fill them to the saturation limit.

The problem is to use these city centers most efficiently by eliminating the less effective use of the available open spaces. For instance, the parking of private automobiles along the curbs interferes with the movement of traffic and thus, first by limitation and then by complete prevention of "parking", the streets are "regulated" to their higher use. Formerly, people catered to the business of the man in the automobile; now they find that the private automobile with its average load of about 1½ passengers per car is using too much of the valuable space in down town streets and they are going to make it so uncomfortable for the owner that he will leave his machine home and come down town on the motor coach, the street car, or the subway. Therefore, the new conception of city planning, which comes with the continued growth of the use of the automobile, is that down town streets are primarily for the use of pedestrians and that passengers through city centers must be handled *en masse*. Much of the business that depends on the private automobile for service or patronage must move to some outlying sub-center where

curb parking is still possible. Ordinarily, this means that when business frontage becomes worth $1 000 per front foot it is subject to traffic restrictions, which will be more and more in favor of the pedestrian and of through traffic and less and less in favor of parking and of the individual in the private automobile.

Decentralization.—Big business has already sensed this movement toward decentralization; branch banks have sprung up on nearly every corner of local business centers, and automobile agencies, accessory supply stores, and oil stations at nearly every "four corners" make convenient shopping centers for the automobile owner. The same automobile which has congested down town streets now can take the owner to a theatre, to a store, to a library, to a dentist, or to some similar place of business or pleasure located in a sub-center.

Thus, the automobile, which is the cause of modern congestion, furnishes the means of escape. The thesis for the design of great Metropolitan Districts, then, becomes "a city center surrounded by satellite sub-centers all interconnected with the best possible transit arrangements."

These sub-centers may even start as an automobile "cross roads" and gradually evolve into satellite sub-centers of no mean importance, but their history is sure to reflect the development of constantly improving transportation facilities. Growth and prosperity are functions of easy access and unrestricted movement. This leads to a consideration of the relation of the automobile to other forms of transportation.

Each Vehicle Has a Place.—In the scheme of transit arrangements each mode of transportation has a place of its own, depending on the density of traffic to be handled, with just sufficient overlapping to make an easy transition from one system to another. Under a given set of conditions, it is possible to prepare curves showing the cost of a passenger-mile, a seat-mile, or a car-mile, which curves will indicate the influence of density on the total cost of operation, including depreciation and interest on the investment. With a private automobile this cost will range from, say, 20 cents to 5 cents per car-mile, depending on conditions of use.

Next in the scale will come the auto-bus or the "motor coach", as people are beginning to call it, with a cost per coach-mile of from 15 to 30 cents, but with the possibility of giving service at the rate of 2 cents per passenger-mile. The trackless trolley is intermediate between the motor coach and the trolley car, which latter vehicle, under favorable conditions, can carry passengers for as low as 1 cent per passenger-mile.

The "rapid transit" car on its elevated structure, in an open cut or in a subway, has the great advantage of capacity as it can run in trains of as many as ten cars, but at no less cost per passenger than the trolley car. For each density of traffic there is a corresponding vehicle for most economical operation so that the transportation system for any given district should be co-ordinated to obtain the best results.

The Place for the Motor Coach.—The private automobile, of course, has no regular place as a commercial carrier in the general transportation plan but

the motor coach has a very definite field, which is defined by the first cost and the total cost of operation.

The outstanding advantage of the motor coach is the fact that its gross earnings per annum may be equal to the original investment. A motor coach costing $10 000 (including repair facilities) can earn $10 000 per year. Ordinarily, a trolley system can earn in one year an amount equal to only about 25% of its total investment and a rapid transit system often has a gross yearly income of less than 12½% of its first cost. Furthermore, the motor-coach system can be added to, unit by unit, as the business grows and without the delay usually experienced in locating and building tracks and designing and constructing power plants, sub-stations, and distributing systems. The motor coach is a vehicle, then, which can be used on the skirmish line of new business. Wherever there is $10 000 gross earnings per year per mile in sight, a bus line can be established giving, say, 30-min. service, and as soon as patronage increases more busses can be added and the headway reduced. One of the important functions of the motor coach, therefore, is to act as a feeder, extending the transit network into otherwise unserved territory.

In the smaller cities motor coaches can run on parallel streets only ¼ mile apart, whereas with a trolley-car system the lines can be maintained only at ½-mile intervals. This development means that in the design of districts it is extremely important to provide through streets every ¼ mile throughout the territory. A population density averaging 5 persons to the acre is potential motor-coach territory, so that the bus may follow the subdivider very closely.

The Limitations of the Motor Coach.—The very advantage which the motor coach possesses in having each unit carry its own overhead charges works as a disadvantage when density of traffic develops. With this system additional units do not reduce the overhead charges on each unit as with a trolley system, where each car must stand its proportion of the costs of the tracks, feeder system, and power plants. The result is that, when good service demands a vehicle every 10 min., or oftener, the motor coach must give way to the trolley car. The cost of carrying a passenger per mile cannot be reduced to much less than 2 cents in a motor coach, whereas a trolley-car system can reduce this cost to as low as 1 cent per passenger-mile, if a certain percentage of the patrons are allowed to stand during the rush hours. At about a 10-min. headway, the two systems overlap and, at this point, the pioneer motor-coach system may well turn over its business to the trolley-car system, and seek other territory to explore.

"De Luxe" Transportation.—The modern motor coach can now take the place of the private automobile on the down town streets, and along the boulevards, to supply transportation at a charge of approximately twice the street-car fare. This kind of service should not be considered as competitive with street cars as it occupies a field which the trolley car does not cover. The obvious place for the electric street car is within a radius of about 5 miles from the center of the city, running through from one side to the other—with one fare (preferably 5 cents) and universal transfers. The motor coach can handle the patronage beyond the 5-mile limit for a fare of 10 cents and

higher for a ride directly to the city center, and such service would be an acceptable substitute for the private automobile which is now handicapped with so many hardships when it reaches the business center.

Street-car systems have been operating at a gross annual income of from $15 to $25 per capita. Considering that many families of four maintain an automobile at a cost of more than $100 per month, or at the rate of $300 per person per annum, at once, there appears the possibility of raising the earning power of the motor-coach system by encouraging the riding habit with better and better service and equipment.

Other Applications of the Motor Coach.—There are many other uses for the motor coach in building up the community. Wherever there are a dozen people desiring transportation to any given place at the same time there will be a use for the public automobile.

Not only may the motor coach be used as a broadcasting agency for the trolley system or for the rapid transit lines, but it can actually be used to parallel existing street-car lines as an auxiliary during rush hours.

Sometimes an isolated satellite community will subsidize busses to give either free rides or reduced fares on a bus line connecting with a stop on the near-by trolley of the interurban line.

District high schools are using motor coaches to collect and distribute their pupils over a large district, thus making higher education available to many families living in the country.

Auto stages are making regular trips over constantly improving roads throughout the country and these independent stage lines are being consolidated into unified systems as rapidly as the advantages of co-ordination are realized. The existing steam and electric railroads are beginning to recognize the value of the automobile, not only as an auxiliary, but as an integral part of a well-balanced system of transportation service to the entire district. This comparatively new vehicle for public transportation promises to carry its fair share of the great increase in passenger and freight movement which grows much faster than the population.

Zoning and Automobiles.—Just as the private automobile enabled many well-to-do people to live in the country and yet enjoy most of the advantages of city life, so the motor coach is now making it possible for all classes of people to own a home in a district zoned for single family residences.

Street-car systems and multi-family residence districts seem to go hand in hand and local commercial centers spring up naturally along the car tracks. Rapid transit lines mean contiguous districts for apartments and business centers filled with skyscrapers. Tracks mean more or less of congestion for a built-up city, whereas the motor coach, with its ability to earn its way and with comparatively a much lighter gross income, can be made the means of escape from the disadvantages of congestion. "Rails", therefore, are the indication of a tendency toward congestion or contraction, while "rubber" is the indication of the opposite tendency toward the open country or expansion. Both mvements need encouragement, and in all zoning regulations, the relation between the public automobile and other forms of transportation should be recognized.

Regional Planning Specifications.—As an example of the application of the fundamental principles of regional planning as affected by the continually increasing use of the automobile, the speaker submits the following requirements which have been adopted and are now being used in the practice of the Los Angeles County, California, Regional Planning Commission.

Based on the highway map in existence as of January, 1923, a regional map has been prepared showing a complete network of "through" roads, which are to provide automobile traffic circulation to every part of the region. This plan provides major highways, not less than 90 ft. (and preferably 100 ft.) in width between property lines about 1 mile apart—often on the section lines in the valley districts.

On the half-section lines or at about halfway between the major highways are laid out intermediate highways 80 ft. wide. A right of way of this width will allow a pavement of 56 ft. between curbs, which is just sufficient for four lines of moving vehicles with a parking space along each curb.

On the quarter-section lines, or at a distance of approximately ¼ mile from the major highways and the intermediate highways are rights of way, 60 ft. wide, for the minor through highways. These streets will provide for pavements 36 ft. in width, allowing for two lines of moving vehicles plus the usual parking space along the curb.

All other streets that can possibly be termed "through streets" are made not less than 60 ft. in width, and only where they are of a strictly local character or in hillside sub-divisions are streets of less than 60 ft. in width approved.

An interesting detail is the requirement of rounded corners which provides for a radius of not less than 30 ft. at the curb line at the intersections of all streets; this arrangement contributes considerably to the capacity and safety of the street.

This highway plan contemplates eventual motor-coach service at least every ½ mile throughout the district. This will naturally serve satellite business centers, scattered through the region at convenient and economical strategic positions. At these sub-centers there should be convenient transfer facilities to the electric rapid transit or to the suburban lines, connecting directly to the main metropolitan business center.

At a short distance from the rapid transit line, or sometimes directly contiguous to it, is a double highway, with a roadway on each side of the track, connecting each sub-center with its neighboring center. These parallel highways provide a right of way for a "local" motor-coach service carrying passengers between the "express" stops on the rapid transit electric lines. This arrangement makes it possible to cut out the local stops on the electric lines and devote the rails to express service with stations about 2 miles apart resulting in the possibility of reaching the periphery of a 20-mile radius by means of "rapid transit" in less than 1 hour from the time of leaving the central station.

Business Sub-Centers.—Actual development of sub-centers shows that, as a rule, each grows up about a trading center, which crystallizes around a

nucleus at some intersection of the major highway system. Therefore, each "four corners" is a potential sub-center and the subdivider very naturally lays out his plot at these intersections as business property. Business lots are also scattered along the major highways (sometimes the entire length of the highway), thus providing many times more business frontage than will be actually needed for years to come.

In Los Angeles County, wherever business is contemplated, the street is to be 100 ft. wide to allow for 70 ft. of pavement and 15 ft. of sidewalk space. Each business block is to have an alley not less than 20 ft. wide to allow for the delivery of parcels and goods from the rear of the stores.

Corner lots are preferably to be of double width to allow for a "cut-off" corner at the street intersection. A radius of 15 ft. at the property line or a diagonal cut-off connecting set-back points 12 ft. from the corner increases the sidewalk space and the visibility sufficiently to justify this requirement.

A Set-Back Line Proposal.—One of the most important highway developments that has been evolved from regional planning activities is the conviction that there must be "set-back" lines to provide for future requirements.

The use of the automobile required streets not less than 100 ft. in width at all business centers. This width will allow for a through traffic strip for four lines of vehicles at the center, and two widths of 15 ft. each along the curbs for local parking; 70 ft. of pavement and 15 ft. of sidewalks is the irreducible minimum for a business street anywhere in the Los Angeles District. Probably the quickest way to realize this requirement is to adopt a regulation that "no structure shall be built nearer than 50 ft. to the center of any through street."

If the automobile is to occupy its proper place in the community as a common carrier, if the convenience of the sub-center is to be encouraged, and if there is any hope of controlling the forces of decentralization which are at hand to offset the disadvantages and even evils of city congestion, the police powers must be wielded to provide for adequate open spaces for the circulation of traffic.

Grade Elimination.—Of equal importance to capacity are the requirements for additional safety. Railroad trunk lines and electric rapid transit rights of way must be separated from the grade of the automobile highways. In the ultimate analysis this means the unification and, very probably, the electrification of transcontinental railroad terminals. The expenditures called for by these improvements are staggering in their totals and can be justified only by the fact that tonnage increases as the square and sometimes as the cube of the population. This growth and anticipated expenditures call for a comprehensive plan and a logical, consistent, step-by-step program. With such a plan and program in mind, the separation of grades on main highways may be begun at once, making each expenditure part of the ultimate result.

It is hard to visualize the conditions which will exist on highways when there are ten times as many automobiles as at present (June, 1924), but it is certain that the automobile traffic will be increased by ten times at no far distant date. "More roads, wider roads, and safer roads" must be the slogan in

this struggle with traffic and congestion; until there is complete grade separation the highways will not be entirely safe.

One of the next big adventures in regional planning will be the construction of safety highways located, perhaps, on rights of way over and through congested districts. Bridges over rivers and railroad tracks in the industrial districts will naturally be elongated to provide safe passage over switch tracks, and such viaducts will point the way to the more extensive use of this kind of an "elevated" for the use of the automobile.

Recapitulation.—The Mobility of the Automobile.—The flexibility of the automobile is the controlling feature which makes it available as an addition to present transit facilities of equal importance with the steam locomotive and the electric car as has already been emphasized.

It can be installed on the "unit" basis and each unit carries within itself its own power plant and its own overhead charges.

The first investment per seat in a "motor coach" to be used as a common carrier is about one-fourth of the investment required by a trolley-car system so that it can be used to advantage as a pioneer for new business in comparatively sparsely settled districts.

It is the only vehicle which does not depend on density of population for its support, in fact as soon as congestion appears the motor coach is ready to turn over its load to be carried on "rails" instead of "rubber". The automobile, therefore, is the exponent of expansion.

Decentralization is the natural result of the congestion caused by the use of the automobile and the same vehicle which causes the congestion of street traffic, furnishes a means of escape, provided enough roads are built to carry it around and away from the city centers.

The very danger which the automobile has brought at grade crossings and on the highways will result in the building of "off-grade" structures which will provide safety highways for the future and which would have been financially impracticable with less density of traffic.

Regional planning itself is the direct result of the use of the automobile which has made time and not distance the limit of modern activities and influence. Any locality within a distance of an hour's automobile ride is an integral part of the Metropolitan Area and its physical improvement is a matter of common concern to every inhabitant in the district. Safety, accessibility, convenience, speed, economy, adequacy, and co-ordination are requirements alike for the steam roads, the electric lines, and the motor-coach systems which provide "mass transportation" to all the people of the great residential districts.

Prosperity is a function of activity. The economic urge toward increased transit facilities has been greatly emphasized by the use of the automobile. These vehicles have been produced much faster than the roads on which they operate. In furnishing transportation service for future generations, or even for the immediate future, "rails" must be loaded to their full carrying capacity and "rubber" must be provided with a network of through roads, all interconnected to avoid congestion in the outlying districts and to provide for the orderly direction of traffic at the city centers.

DISCUSSION

STEPHEN CHILD,* M. AM. SOC. C. E.—Landscape architects, while basing their work on scientific data, make every effort to keep art and beauty constantly in mind. They maintain that the useful and the beautiful are the two blades of a pair of scissors—that both are equally essential to good work. Too often the engineer has striven to do with one blade what would have been better done with two and has thereby produced a botched job. To illustrate, for ten years the speaker was Assistant City Engineer of Newton, Mass., and during that time an important boulevard system was laid out extending across the city from the Boston line to the Charles River. This Commonwealth Extension has two roadways and a grassed, tree-shaded, central strip reserved for the trolley car lines.

When it came to the planting of the trees, only one blade of the pair of scissors was used, for several miles of that central parking space were planted with a double row of Rock Maples. These, as they grew up, assumed an oval outline, with the result that an ugly gash had to be made in every tree to permit the trolley car to pass. The central trolley section should have been planted with elm trees or any others that would naturally arch over the strip without necessitating their mutilation. The point to be emphasized is that unless the beauty as well as the convenience of the community receives attention bad city planning will result.

The writer heartily agrees with the author in his suggestions as to decentralization and helping to promote the establishment of satellite towns. There is one suggestion that may be criticized—Mr. Damon recommends having a radius of the property line at street corners of 15 ft. and the radius of the curve of the curb line as great as 30 ft., and yet he speaks feelingly of managing traffic to help pedestrians. In the arrangement suggested, a pedestrian must go around that 15-ft. turn and cross the paving where it is narrowest. He ought not to blunder ahead and cross in the middle of the curve where he would be meeting the traffic at an acute angle. One of the fundamental principles of controlling traffic is to arrange it so the streams cross at right angles. A 15-ft. radius for curves at property lines and a 30-ft. radius for the curves at the curb line is too generous an allowance. It makes the pedestrian either take extra steps or extra risks.

In the business center of Los Angeles, Calif., there are the north and south streets—Spring Street, Main Street, Broadway, and others—crossed at right angles by east and west streets—Fifth Street, Sixth Street, and others. Until a better solution can be evolved, the speaker's suggestion is that the traffic be restricted in alternate streets of this super-congested district, Main Street, for example, to one direction only and Spring Street to the other direction and likewise with Fifth and Sixth Streets. Would not this reduce congestion at the corners about one-half? This is not put forward as the best or a permanent solution, but as something to be tried as a temporary measure.

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In Palo Alto, there is a complicated transportation problem with the two factors of auto busses and trolley cars, much in evidence in the discussions. The main line of the Southern Pacific Railroad runs practically north and south, dividing the town into two main parts—on the east side, the village with about 7 000 population and on the west side, nearly a mile away, Stanford University, with a population of approximately 3 000.

The Peninsula Street Railroad has a double-track line running west from the station on University Avenue—the main street—for about ½ mile, where the tracks branch, one running south for about 1 mile and another continuing straight on in University Avenue for 1 mile or more. From the railroad station on the ½ mile of double track there is 10-min. service, which is Mr. Damon's limit for effective trolley car service. He claims that for a service of 15 or 20-min. interval, motor busses are more efficient. On the single-track streets, the cars run on 15-min. intervals.

The problem is complicated by the fact that University Avenue needs paving badly. The Street Railroad Company has refused to pay its share of this repaving, taking the ground that it cannot afford it; it has actually gone before the Railroad Commission and asserted that it would rather give up its franchise than do it. The speaker is informed that a similar state of affairs exists in several other towns. The problem is, would it be better to abolish these unprofitable street railways and have auto busses?

The introduction of this kind of service has a bearing on another factor—the grade-crossing problem. As to this, perhaps the only feasible proposition is to raise the railroad tracks 3 or 4 ft. and depress University Avenue enough to pass under them. If the trolley lines are maintained, there must be a subway broad enough to accommodate at least three lines of traffic, one for the trolley and one each way for other traffic. If, however, the trolley is abolished, there would be need for a width that would accommodate only two lines of traffic, the auto busses could use these lines as well as other vehicles.

There is another complication, however, Stanford University, nearly a mile west of the railroad, depends on the railroad for freight accommodations for many kind of supplies, oil, coal, etc. The University has an agreement with the Trolley Company and the Railroad Company whereby, at convenient hours, freight and tank cars are moved over the trolley track to the University buildings and the University authorities are very urgent that this arrangement be maintained.

No solution for the problem has yet been determined, but these details are mentioned to show how complicated such a matter may be even in a small town and how difficult it is to solve it on general principles, even such as are so clearly laid down by Mr. Damon.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).—It is difficult indeed, as the author states, to visualize highway conditions when there will be ten times as many automobiles as at present; to the writer such a number is inconceivable. Even in California where motor cars are very numerous this density has not been approached. The writer believes that

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the point of saturation as to numbers is even now in sight, although traffic will continue to increase rapidly with increased improved highway mileage. Be this as it may, Mr. Damon has indicated clearly and correctly the influence of the automobile and motor coach on regional planning and zoning.

One feature of the general problem not discussed is that of metropolitan park reservations. In former years these reservations were made within ten miles of the centers of great cities; to-day, the distance may be quadrupled. With increased distance come increased possibilities in area, use, and scenic advantages. In an almost uncanny manner the Federal Government sensed the needs of the future and established National Parks long before the advent of the motor tourist. At present, the desirability of purchasing smaller tracts, especially in the Eastern States, needs no demonstration. While park reservations are being secured, future metropolitan water supplies can be guaranteed and game and forest preserves established.

An interesting detail of this excellent paper is the brief treatment of corner rounding to facilitate traffic. The 30-ft. curb radius will effect the end desired, but in the writer's opinion is much too great for general practice. The radius to be adopted depends on the width of street, density of traffic, location of cross-walk, and "cut-off" from the corner property to provide visibility. In the matter of corner rounding the interests of motorist and pedestrian are diametrically opposed. With a large radius the possible speed of motor vehicles in turning the corner is greatly increased, but this is accompanied by increased danger to the pedestrian; with a small radius, motor traffic is throttled with less peril to those on foot. Excessively rounded corners encourage excessive speeds which, in turn, can only be checked by expensive police traffic regulation.

Theoretically, the curb at street crossings should be normal to the direction of pedestrian travel. This is not entirely possible when the cross-walk and sidewalk are continuous, as is the usual practice. To avoid this obliquity due to the curvature of the curb corner, it is possible to move the cross-walk back toward the prolonged property line to which it is parallel. Where curb and cross-walk are not mutually perpendicular there will be difficulty and even danger in stepping to and from the gutter. Slippery corner bars in concrete radius curbing at cross-walks may be a source of serious danger in wet weather. There are no definite rules for determining the corner radius; the engineer's judgment must be based on the conditions as he finds them in the light of the principles herein set forth.

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Paper No. 1571

THE DESIGN OF A MULTIPLE-ARCH SYSTEM AND PERMISSIBLE SIMPLIFICATIONS*

By A. C. Janni,† M. Am. Soc. C. E.

With Discussion by Messrs. J. Charles Rathbun, D. B. Steinman, George Paaswell, Edward Godfrey, E. H. Harder, Hardy Cross, Harold B. Hammill, George E. Beggs, C. W. Hudson, A. B. Cohen, T. D. Mylrea, Charles S. Whitney, and W. Schachenmeier.

Synopsis

A strictly theoretical design of a multiple-arch system the arches of which cannot be considered as having fixed ends, is hardly practical. It is the aim of this paper to show whether, with such a system, it is possible, for practical purposes, to simplify or shorten the strictly theoretical method. From the results obtained, it seems that some simplifications are quite permissible.

General

The design of an elastic arch, no matter whether analytical or graphical, or a combination of both, represents one of the most complex problems in engineering. Formerly, relatively few arches were built, but, recently, especially since the advent of reinforced concrete as a material for construction, the rapid increase of road building has necessitated more bridges and the arch span has become the type most generally used. Often, due to physical conditions, a series of arches is required, in which case, the piers supporting this system of arches may be of such dimensions that they cannot correctly be assumed as unyielding under the action of loads.

* Presented at the meeting of September 3, 1924.

† Cons. Engr., New York, N. Y.

The case in which an arch may be considered as having unyielding supports—as with a single span founded on rock, or with an arch system, using piers of such dimensions that each arch may be considered as having fixed ends—has been treated by theory in several ways; in recent years, there has been added another method, purely graphical,* that is founded exclusively on the theory of the Ellipse of Elasticity.

This same theory also lends itself to the more complex problem of an arch the supports of which are yielding. It will be applied to a practical case, in which three arches, A_1, A_2, and A_3, have been considered as supported by two rather slender piers, P_1 and P_2 (Plate III).

For the sake of clearness, this problem has been treated, first, by considering A_2, one of the three arches, as having fixed ends, making the design accordingly, and obtaining the influence lines of the moments; and, second, by considering Arch A_2 as being between two yielding supports, the one on the left being the combination, A_1, P_1, the second being A_3, P_2. The influence lines, in this case, have been computed for the three arches, and have been plotted, together with the influence lines obtained in the first case, so that the difference of moments affecting the same sections under both assumptions may be seen at a glance.

Also, for the sake of clearness, Plates III, IV, and V are shown in light and heavy lines; the light lines refer to the case of Arch A_2 on unyielding supports; and the heavy lines refer to the assumption of Arch A_2 on yielding supports, and to all variations of moments and stresses acting on this system and arising from the assumption made.

Design Assuming Unyielding Supports

Plate III shows half an arch and its corresponding supporting pier. For convenience and clearness, a summary of the procedure will first be given under the assumption that the arch has fixed ends.

The geometrical axis has been divided into ten equal parts, each part being 2.587 ft. long, and the center of each voussoir has been located. The semi-axis of the ellipse of elasticity of each voussoir, along its geometrical axis, has been computed by the formula: Major semi-axis $= \rho = \sqrt{\frac{l^2}{12}} = 0.748$ ft., l being the length of the voussoir. Then, the moment of inertia, I, about the horizontal gravity axis, the area, A_1, A_2, etc., and the minor semi-axis of the ellipse of elasticity, P_1', P_2', etc., were computed for each voussoir. The results are given in Table 1.

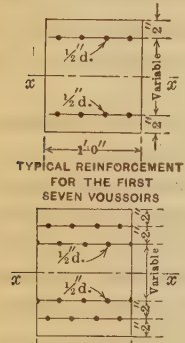
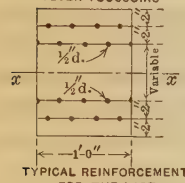
From the quantities computed in Table 1, the elastic weights of the voussoirs were found, as shown in Table 2.

Polygon p_1.—The elastic weights were laid off, at a convenient scale, vertically in order, as shown on Plate III, and a pole distance assumed equal to twice the total sum of these elastic weights, giving the polygon of forces with a pole, P_1. Verticals through the centers of gravity of the ten voussoirs

* "Reinforced Concrete Construction," by George A. Hool, Vol. III, Chapter 8.

were drawn, and, considering each elastic weight as a force applied vertically, at the center of gravity of the voussoir, the funicular polygon was drawn.

TABLE 1.—Elements of Arch Sections.

	I, in feet.⁴	Area, in square feet.	ρ', in feet.
 TYPICAL REINFORCEMENT FOR THE FIRST SEVEN VOUSSOIRS	$I_1 = 0.101$	$A_1 = 1.162$	$\rho_1' = 0.294$
	$I_2 = 0.107$	$A_2 = 1.182$	$\rho_2' = 0.300$
	$I_3 = 0.113$	$A_3 = 1.202$	$\rho_3' = 0.306$
	$I_4 = 0.120$	$A_4 = 1.222$	$\rho_4' = 0.313$
	$I_5 = 0.127$	$A_5 = 1.242$	$\rho_5' = 0.319$
	$I_6 = 0.134$	$A_6 = 1.262$	$\rho_6' = 0.326$
	$I_7 = 0.150$	$A_7 = 1.302$	$\rho_7' = 0.339$
	$I_8 = 0.187$	$A_8 = 1.524$	$\rho_8' = 0.350$
	$I_9 = 0.217$	$A_9 = 1.584$	$\rho_9' = 0.370$
 TYPICAL REINFORCEMENT FOR THE LAST THREE VOUSSOIRS	$I_{10} = 0.249$	$A_{10} = 1.644$	$\rho_{10}' = 0.389$

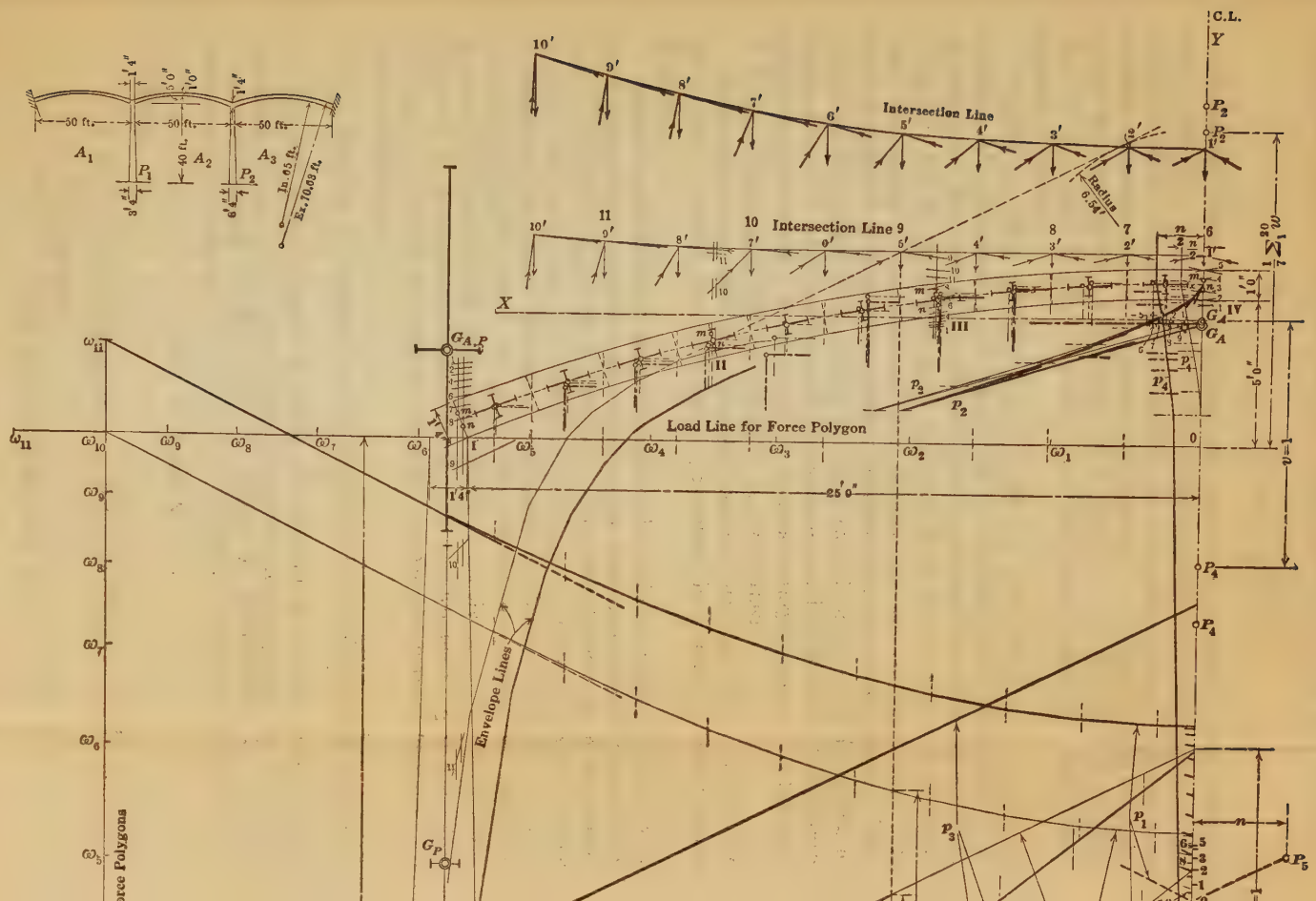
It is to be noted, once for all, that the scale for the elastic weights, as shown on Plate III, is quite independent of the scale of the drawing.

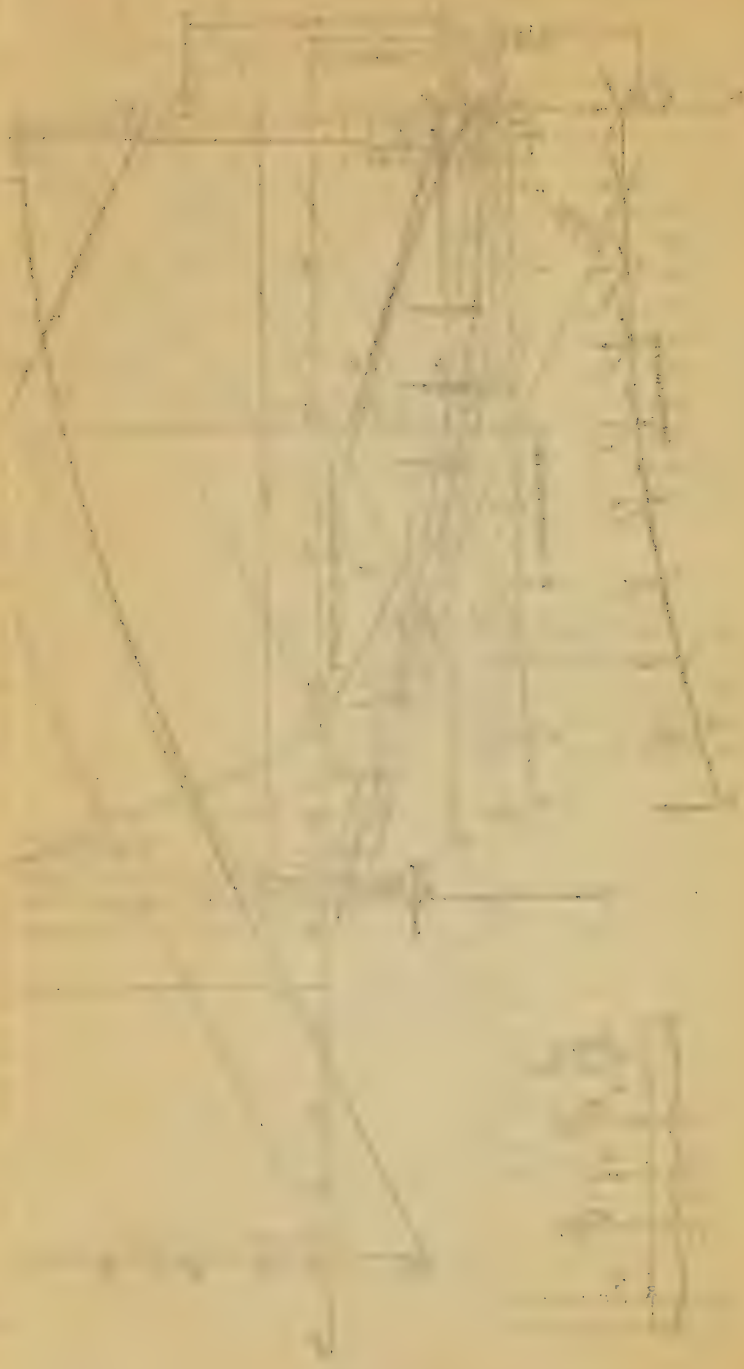
Any vertical force applied to the arch will induce a moment equal to the product of the force and the ordinate that this polygon intercepts on the line of the applied force. For this reason, Polygon p_1 is called the Polygon of Moments.

TABLE 2.—Elastic Weights of Arch Sections.

For $E = 1.00$:		(*Brought forward*) = 114.29	
$\frac{\Delta s}{E I_1} = \frac{2.58}{0.101} = 25.54$		$\frac{\Delta s}{E I_6} = \frac{2.58}{0.134} = 19.25$	
$\frac{\Delta s}{E I_2} = \frac{2.58}{0.107} = 24.11$		$\frac{\Delta s}{E I_7} = \frac{2.58}{0.150} = 17.20$	
$\frac{\Delta s}{E I_3} = \frac{2.58}{0.113} = 22.83$		$\frac{\Delta s}{E I_8} = \frac{2.58}{0.187} = 13.79$	
$\frac{\Delta s}{E I_4} = \frac{2.58}{0.120} = 21.50$		$\frac{\Delta s}{E I_9} = \frac{2.58}{0.217} = 11.88$	
$\frac{\Delta s}{E I_5} = \frac{2.58}{0.127} = 20.31$		$\frac{\Delta s}{E I_{10}} = \frac{2.58}{0.249} = 10.36$	
Total			186.77

Polygon p_2.—The same elastic weights were laid off in order horizontally, and a pole distance, P_2, was taken. This pole distance, for convenience (as will


C.L.
Y
X
Intersection Line
Intersection Line 9
Radius
6.54'
Load Line for Force Polygon
Envelope Lines
50 ft.
50 ft.
50 ft.
40 ft.
A_1
A_2
A_3
P_1
P_2
In 45 ft.
Ex. 70.03 ft.
G_A
G_P
G'_A
25' 0"
I
II
III
IV
ω_{11}
ω_{10}
ω_9
ω_8
ω_7
ω_6
ω_5
ω_4
ω_3
ω_2
ω_1
0
p_1
p_2
p_3
p_4
P_4
P_5
n
m
Force Polygons



be seen later) was taken as an aliquot part, ξ $\left(\text{assumed as } \frac{1}{7}\right)$ of $2\ \Sigma_1^{10}\ w$, that is:

$$\xi\ .\ 2\ \Sigma_1^{10}\ w = \xi\ .\ \Sigma_1^{20}\ w = \frac{1}{7}\ \Sigma_1^{20}\ w.$$

Horizontal lines passing through the centers of gravity of the voussoirs were drawn, and assuming that these horizontal lines are lines of action of the elastic weights now acting horizontally, the funicular polygon, p_2, was constructed, corresponding to the polygon of forces, P_2. The last side of Polygon p_2 cuts the vertical axis of symmetry, $Y\ Y$, of the arch at the center of gravity, G_A, of the system of elastic weights considered as acting at the appropriate centers of gravity.

Polygon p_3.—The next step is to locate the anti-pole* for the ellipse of elasticity of each voussoir with respect to the vertical axis of symmetry, $Y\ Y$, of the arch. In Plate III, these anti-poles have been located for the first six voussoirs; for the remaining four voussoirs, they would practically coincide with the center of the corresponding ellipse, and are so considered. The statical moments of the elastic weights, applied vertically at the centers of the voussoirs, the amount of each being found by extending the sides of Polygon p_1 to the axis, $Y\ Y$, have been considered as forces applied vertically to the anti-poles found previously. With a pole, P_3, at an arbitrary distance from the axis, $Y\ Y$, a corresponding funicular polygon, p_3, was drawn.

This polygon, p_3, will determine the vertical reactions of the arch for a given load. For this reason Polygon p_3 is called the Polygon of the Vertical Reactions.

Polygon p_4.—Similarly, the anti-poles of the various ellipses of elasticity of the voussoirs were located with respect to the horizontal passing through the center of gravity, G_A, of the elastic arch. The statical moments of the elastic weights, with respect to the horizontal through G_A, given in amount by the extensions of the sides of Polygon p_2 cutting the horizontal through G_A, have been applied horizontally to the anti-poles previously mentioned, and, assuming a pole, P_4 (Plate III), the corresponding funicular polygon, p_4, has been drawn. (The Pole P_4 was selected on the vertical, $Y\ Y$, at a distance from the horizontal through G_A equal to the vertical distance between the parallel first and last sides of Polygon p_3.)

The distance, $\frac{n}{2}$, between the first and last sides of this funicular polygon is proportional to the moment of inertia of the half arch with respect to the horizontal through G_A. After constructing these four polygons, the axis of the ellipse of elasticity of the arch can readily be determined. In fact, the distances of the tangents (vertical and horizontal) to this ellipse are given by:

$$\sqrt{\xi\ .\ v\ .\ n} \text{ and } \sqrt{\lambda_3\ .\ v}$$

in which,

$\xi = \frac{1}{7}$ = the aliquot part of the pole distance, $2\Sigma_1^{10}\ w$, already mentioned;

* See Appendix, Paragraph 7, *et seq.*

$v = 8.32$ = pole distance, P_4;

$n = 1.46$ = the part of the horizontal through G_A intercepted between the first and last sides of Polygon p_4; and

$\lambda_3 = 20.00$ = pole distance of Polygon p_3.

As the arch is symmetrical, the axes of this ellipse are horizontal and vertical, and, therefore, the distances just obtained give the values of the two semi-axes, that is:

$$\rho = \sqrt{\frac{\xi \times \Sigma_1^{20}\, w \times v \times n}{\xi \times \Sigma_1^{20}\, w}} = \sqrt{\xi \times v \times n} = \sqrt{\frac{1}{7} \times 8.32 \times 1.46} = 1.30 \text{ ft.}$$

$$\rho' = \sqrt{\frac{\Sigma_1^{20}\, w \times \lambda_3 \times v}{\Sigma_1^{20}\, w}} = \sqrt{\lambda_3 \times v} = \sqrt{20.00 \times 8.32} = 12.90 \text{ ft.}$$

These axes are not shown on Plate III for lack of room, but appear on Plate V.

Polygon p_5.—The horizontal statical moments of the elastic weights about the horizontal through G_A, were plotted on the vertical, $Y\,Y$ (Plate III), the pole, or center, P_5, being taken at a distance, n, from the vertical, $Y\,Y$. These statical moments, considered as vertical forces, were applied to the anti-poles of the various ellipses of elasticity with respect to the horizontal through G_A. The funicular polygon, p_5, was then drawn corresponding to Pole P_5.

Polygons p_1, p_3, and p_5 are the characteristics of the arch and by means of them its design is readily accomplished. It is seen that these polygons are quite independent of the loading, being determined solely by the geometry of the arch; therefore, they hold good for any hypothesis of loading.

On account of the great importance of these three polygons, it is advisable that the graphical construction be checked by simple computations, according to the following arrangement:

w.	x.	y.	x'.	y'.	x''.	$\frac{w\,x}{\Sigma\, w}$	$\frac{w\,y}{\xi\,\Sigma\, w}$	$\frac{w\,x}{\Sigma\, w} \cdot \frac{x'}{\lambda_3}$	$\frac{w\,y}{\xi\,\Sigma\, w} \cdot \frac{y'}{v}$	$\frac{w\,y}{\xi\,\Sigma\, w} \cdot \frac{x''}{n}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
..	..	..	..	..	..	....	....	....	....	....
..	..	..	..	..	..	....	....	....	....	....

in which,

w = elastic weight;

x = distance from the center of the voussoir to the axis, Y;

y = distance from the center of the voussoir to the axis, X;

x' = distance between the axis, Y, and the anti-pole of the ellipse of elasticity of the voussoir with respect to the axis, Y;

y' = distance between the axis, X, and the anti-pole of the ellipse of elasticity of the voussoir with respect to the axis, X;

x'' = distance from the axis, Y, to the anti-pole of the voussoir with respect to the axis, X.

It is to be noted that λ_3 is the pole distance for Polygon p_3, and that v and n (Plate III) are, respectively, the summation of Columns (9) and (10). All these distances may be measured to any scale. The elastic weights may be taken as referred to any unit.

Line of Intersection and Envelope Lines.—To recapitulate, Polygon p_1 gives the moment of any force applied to the arch, namely, the product of the force and the segment intercepted on the vertical of the force by the sides of this polygon.

Polygon p_3 gives the right and left reactions at the top of piers, on the basis of the vertical distance between the first and last sides of this polygon as a unit; the amount of vertical force is multiplied by the intercept of this polygon on the vertical of the force applied.

Finally, Polygon p_5 gives the horizontal thrust in the arch due to the same force, which thrust is the product of the force applied and the intercept determined by this polygon on the vertical of the force to the same scale as in Polygon p_3. In other words, if a vertical force is applied at any point of the arch and its line of action extended through Polygons p_1, p_3, and p_5, its intercepts on these polygons, measured to the proper scale, and multiplied by the force applied, will give respectively, the moment (defined subsequently), the vertical reactions at the supports, and the horizontal thrust. This difference in the scales in measuring the intercepts—that is, moment intercept to the scale of the drawing, left and right reactions, and horizontal thrust to a scale having as a unit the vertical distance between the first and last sides of Polygon p_3—seems at first to be confusing, but in the progress of the design will prove a substantial simplification of the problem, and avoid a great deal of computation.

In Plate III, ten different positions of the unit load, 1′, 2′, 3′, 4′, 5′, 6′, 7′, 8′, 9′, and 10′, have been assumed. It will be noted that these positions have been selected as having their verticals passing through the center points of the joints of the various voussoirs. This has been done for greater accuracy, because Polygon p_1 and, approximately, Polygons p_3 and p_5, envelop the curves into which these polygons would transform if the length of each voussoir were infinitesimal.

The intersection line, as well as the envelope lines, may be obtained by several methods; the one generally followed in this case is shown in full for Load 5. The intercept between the sides of Polygon p_1, marked $M_{5'}$, is 11.85; therefore, the moment is $M_{5'} = 1 \times \eta_o = R_{A_{5'}} \times \delta = 1 \times 11.85$.

The left reaction, $R_{A_{5'}}$ is determined by the construction shown on Plate III, and its value is the quotient of its length divided by the distance between the first and last sides of Polygon p_3, both measured to the scale of the diagram. That is,

$$\delta = \frac{1 \times 11.85}{\frac{15.10}{8.32}^*} = 6.54 \text{ ft.}$$

* 8.32 is the length of the vertical intercept between the first and last sides of Polygon p_3 measured to the scale of Plate III.

Therefore, if, with the center at G_A, a circle is described, having this quantity as a radius, and if a tangent to this circle is drawn parallel to the reaction, $R_{A5'}$, this tangent will represent the position of the left reaction due to Load 5′. In a similar manner, the position of the right reaction, due to the same load, may be determined.

It is clear that the two reactions and the vertical line of action of the load must meet in a point; a series of such points forming a locus or intersection line is shown on Plate III. The reactions left and right, due to Load 5′, tangent to the envelope line, are also shown.

These diagrams (Plate III), are the fundamental part of the arch design, and form a basis for all further computations. Therefore, they should be carefully drawn to as large a scale as possible and checked numerically.

Influence Lines.—Influence lines have been computed for the four sections (numbered I, II, III, and IV) and for loads from Position I to Position II. For each section, the kernel points, m and n, have been determined, and the verticals through these points drawn. Thus, there are two influence lines for each section, each line being referred to the kernel moment instead of to the center of gravity moment. This method is convenient, for it gives the two moments at each edge of the cross-section directly.

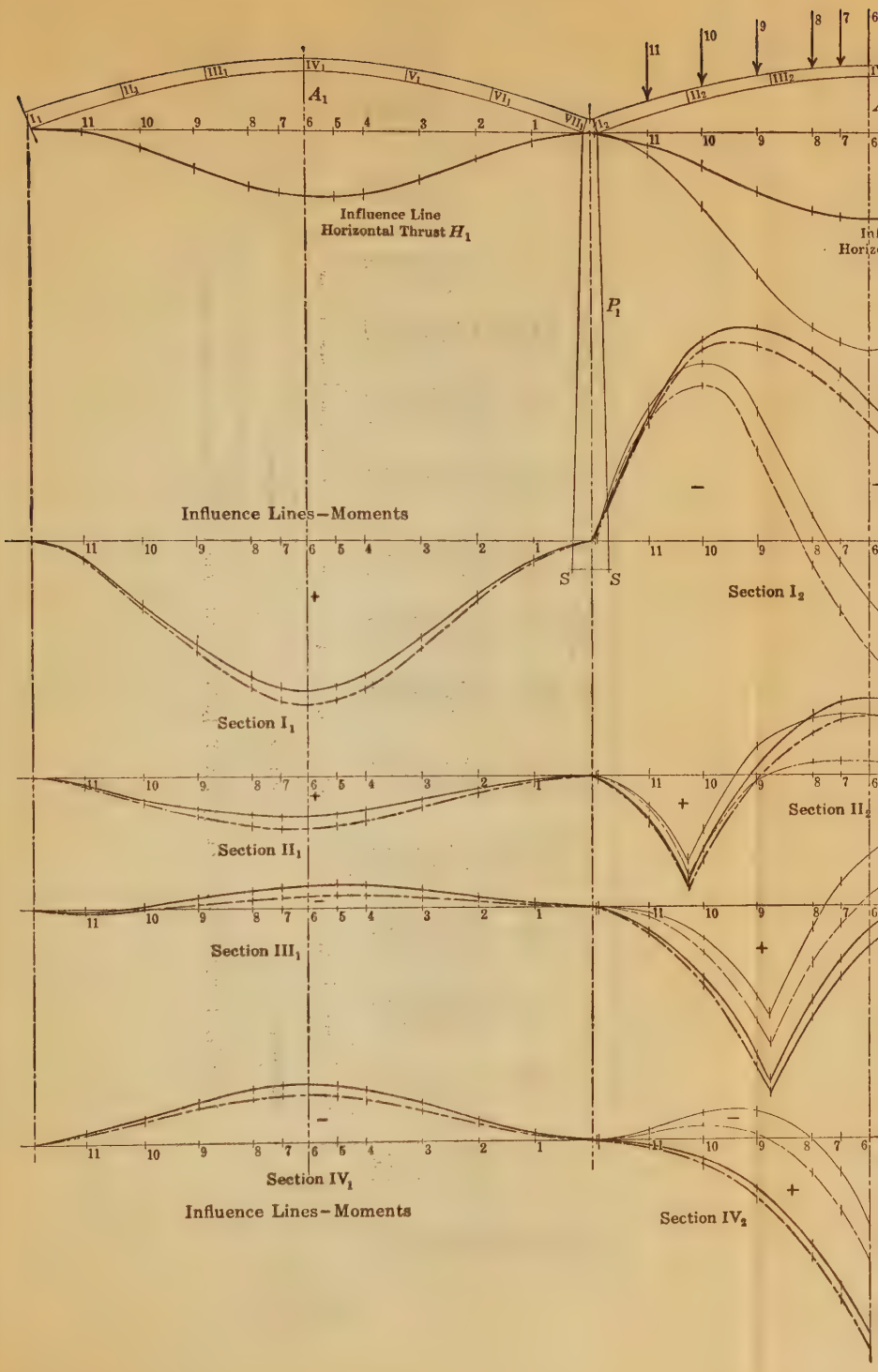
By measuring to scale the intercepts of the verticals through the kernel points between the kernel points and points where the reaction cuts them, and multiplying each intercept by the horizontal thrust (from Polygon p_5), Table 3 has been computed. In Table 3, h_m is the vertical distance from kernel point, m, to the line of the reaction, and M_m is the moment of the reaction about Point m, being the product of h_m and the horizontal component, H, of the reaction. Similarly, h_n and M_n give the vertical distance and moment for the kernel point, n.

It is to be noted that, for Sections II and III, the loads have been placed also on the verticals through the kernel points, m and n; this has been done in order to determine exactly the cuspidal point of each of these influence lines. The results are plotted in Plate IV.

Design Assuming Elastic Supports

If the arch were to be regarded as having fixed ends, then the design, as far as live load is concerned, would stop here, and the computation of stresses in the sections would be easily made from the influence lines already found. In this case, however, there is reason, as will be shown subsequently, to go further in the research, in order to ascertain the actual stresses in the arch. As stated previously, this arch is not rigid at its ends, and the dimensions of the piers are such as to alter materially the distribution of the internal stresses induced in the arch already found. It is for this reason that the piers, as well as the adjacent arches, must be factors in the investigation of stresses in the central arch.

Design of Piers.—For this investigation, Pier P_1 is divided into ten parts, as shown in Fig. 1; for each part, the center of gravity, the ellipse of elasticity, and the elastic weight, have been determined as in the arch. Also, the ellipse


Influence Line
Horizontal Thrust H_1
A_1
P_1
Influence Lines—Moments
Section I_1
Section I_2
Section II_1
Section II_2
Section III_1
Section IV_1
Influence Lines—Moments
Section IV_2
S
S

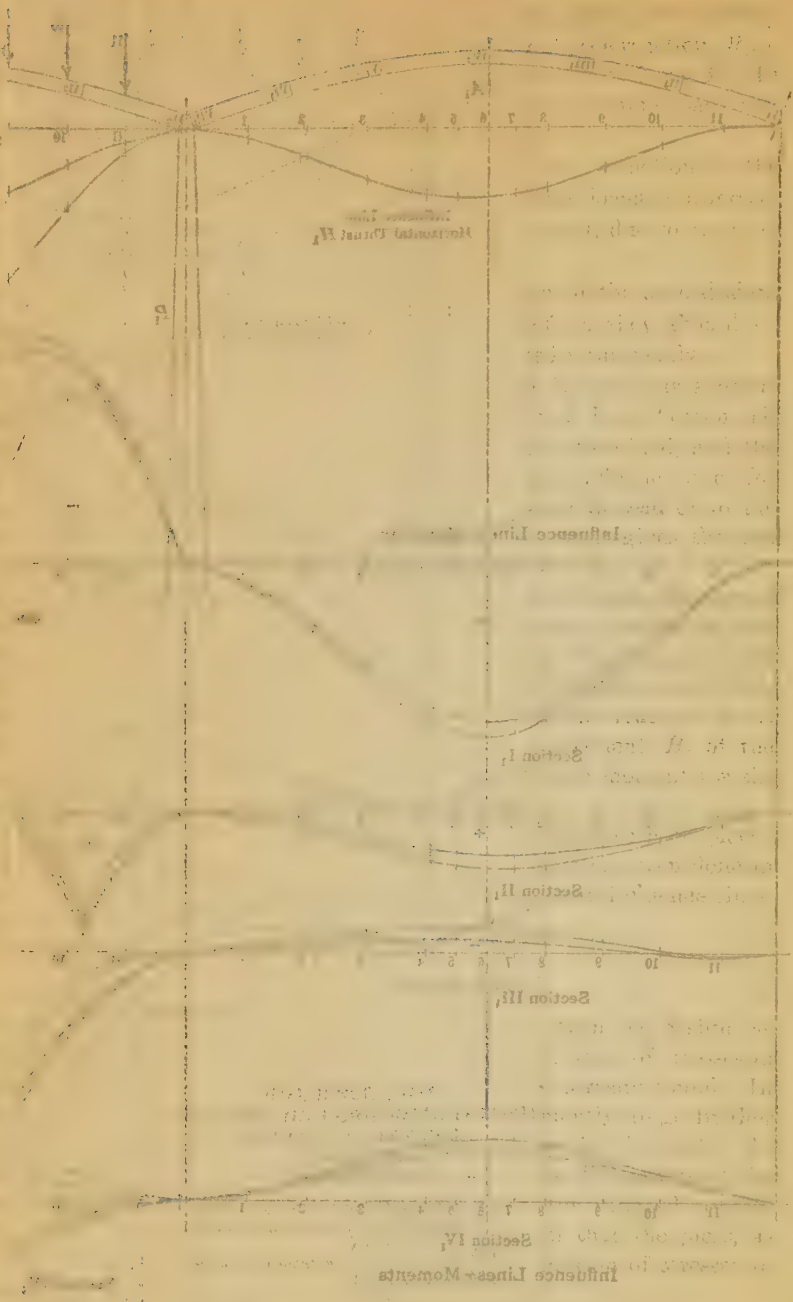


TABLE 3.—MOMENTS AT KERNEL POINTS FOR ARCH A_2 (WITH FIXED ENDS).

Position of load.	H.	Section I.				Section II.				Section III.				Section IV.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.247	+ 1.68	+ 2.12	+0.414	+0.523	−0.34	+0.08	−0.083	+0.019	−1.10	−0.72	−0.271	−0.177	−0.88	−0.51	−0.217	−0.125
2	0.867	+ 1.53	+ 1.95	+1.326	+1.690	−0.37	+0.07	−0.320	+0.060	−1.01	−0.62	−0.875	−0.537	−0.69	−0.33	−0.598	−0.286
3	1.631	+ 1.32	+ 1.75	+2.152	+2.854	−0.41	+0.01	−0.668	+0.016	−0.90	−0.55	−1.467	−0.897	−0.40	−0.05	−0.652	−0.081
4	2.253	+ 1.06	+ 1.50	+2.388	+3.379	−0.48	−0.06	−1.081	−0.135	−0.75	−0.40	−1.689	−0.901	+0.02	+0.36	+0.045	+0.811
5	2.425	+ 0.84	+ 1.29	+2.037	+3.128	−0.50	−0.09	−1.212	−0.218	−0.63	−0.25	−1.527	−0.606	+0.32	+0.68	+0.776	+1.649
6	2.507	+ 0.54	+ 1.00	+1.353	+2.507	−0.55	−0.13	−1.378	−0.325	−0.45	−0.08	−1.128	−0.200	+0.79	+1.14	+1.980	+2.857
7	2.425	+ 0.20	+ 0.66	+0.485	+1.600	−0.57	−0.13	−1.382	−0.315	−0.20	+0.20	−0.485	+0.485	.....	.....	.....	.....
8	2.253	− 0.24	+ 0.25	−0.540	+0.563	−0.57	−0.10	−1.284	−0.225	+0.25	+0.60	+0.563	+1.351	.....	.....	.....	.....
	1.800	.....	.....	.....	.....	.....	.....	.....	.....	+1.40	.....	+2.520	.....	.....	.....	.....	.....
	1.814	.....	.....	.....	.....	.....	.....	.....	.....	.....	+1.76	.....	+3.192	.....	.....	.....	.....
9	1.631	− 1.85	− 1.36	−3.017	−2.218	−0.40	+0.10	−0.652	+0.163	+1.26	+1.62	+2.065	+2.642	.....	.....	.....	.....
10	0.867	− 4.70	− 4.09	−4.074	−3.546	+1.53	+2.05	+1.326	+1.777	+0.81	+1.20	+0.702	+1.040	.....	.....	.....	.....
	0.687	.....	.....	.....	.....	+2.90	.....	+1.992	.....	.....	.....	.....	.....	.....	.....	.....	.....
	0.706	.....	.....	.....	.....	.....	+3.30	.....	+2.329	.....	.....	.....	.....	.....	.....	.....	.....
11	0.247	−12.08	−10.96	−2.971	−2.707	+2.73	+3.11	+0.674	+0.768	+0.60	+0.95	+0.148	+0.234	.....	.....	.....	.....

of elasticity of the whole pier has been determined and its axis computed by the formula:

$$\rho = 4{,}0 \times \sqrt{\frac{1}{12}} = 4 \times 0.289 = 1.156 \text{ ft.}$$

The values of the quantities required are given in Tables 4 and 5.

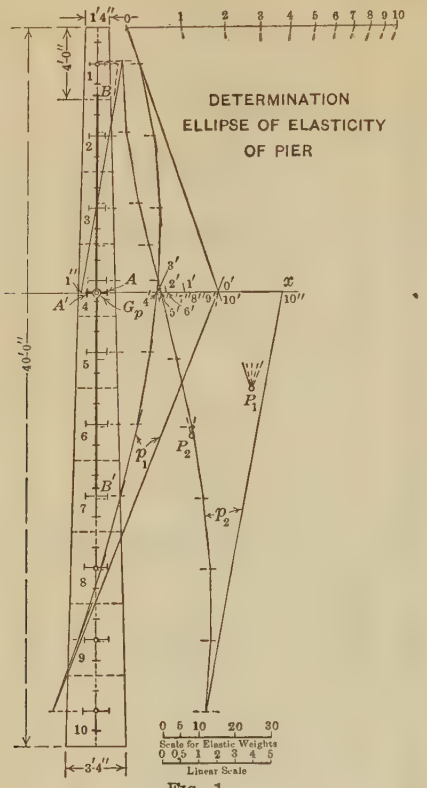
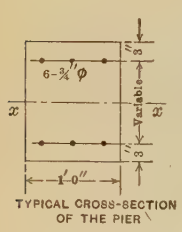


FIG. 1.

TABLE 4.—ELEMENTS OF PIER SECTIONS.



I, in feet.⁴	Area, in square feet.	ρ', in feet.
$I_1 = 1.156$	$A_1 = 1.65$	$\rho_1' = 0.40$
$I_2 = 0.88$	$A_2 = 1.75$	$\rho_2' = 0.42$
$I_3 = 0.40$	$A_3 = 1.85$	$\rho_3' = 0.45$
$I_4 = 0.48$	$A_4 = 1.95$	$\rho_4' = 0.49$
$I_5 = 0.58$	$A_5 = 2.05$	$\rho_5' = 0.53$
$I_6 = 0.68$	$A_6 = 2.15$	$\rho_6' = 0.56$
$I_7 = 0.79$	$A_7 = 2.25$	$\rho_7' = 0.59$
$I_8 = 0.91$	$A_8 = 2.35$	$\rho_8' = 0.62$
$I_9 = 1.05$	$A_9 = 2.45$	$\rho_9' = 0.65$
$I_{10} = 1.20$	$A_{10} = 2.55$	$\rho_{10}' = 0.68$

TABLE 5.—Elastic Weights of Pier Sections.

For $E = 1.00$:		(*Brought forward*) = 52.15	
$\frac{\Delta s}{E I_1} = \frac{4.00}{0.27} = 14.81$		$\frac{\Delta s}{E I_6} = \frac{4.00}{0.68} = 5.88$	
$\frac{\Delta s}{E I_2} = \frac{4.00}{0.33} = 12.12$		$\frac{\Delta s}{E I_7} = \frac{4.00}{0.79} = 5.06$	
$\frac{\Delta s}{E I_3} = \frac{4.00}{0.40} = 10.00$		$\frac{\Delta s}{E I_8} = \frac{4.00}{0.91} = 4.39$	
$\frac{\Delta s}{E I_4} = \frac{4.00}{0.48} = 8.33$		$\frac{\Delta s}{E I_9} = \frac{4.00}{1.05} = 3.80$	
$\frac{\Delta s}{E I_5} = \frac{4.00}{0.58} = 6.89$		$\frac{\Delta s}{E I_{10}} = \frac{4.00}{1.20} = 3.33$	
Total.....			74.61

It is to be noted that while for the vertical axis of the ellipse of elasticity of the whole pier it has been possible to apply horizontally the fractional elastic weights, each at its corresponding part of the pier, for the determination of the horizontal axis of the same ellipse, this construction has not been possible, because the centers of the ten sub-divisions lie in the same vertical. In this case, the analytical method has been followed, with the results shown in Table 6.

TABLE 6.—Moments of Inertia of Elastic Weights of Sections of Pier About the Vertical Axis of the Pier.

Point (1).....	$14.8 \times 0.40^2 = 2.368$	Point (6).....	(*Brought forward*) = 10.457 $5.8 \times 0.56^2 = 1.818$
" (2).....	$12.1 \times 0.42^2 = 2.134$	" (7).....	$5.0 \times 0.59^2 = 1.740$
" (3).....	$10.0 \times 0.45^2 = 2.025$	" (8).....	$4.4 \times 0.62^2 = 1.691$
" (4).....	$8.3 \times 0.49^2 = 1.992$	" (9).....	$3.8 \times 0.65^2 = 1.605$
" (5).....	$6.9 \times 0.53^2 = 1.938$	" (10).....	$3.3 \times 0.68^2 = 1.525$
Total.....			18.886

Having completed the computations shown in Table 6, and knowing that the pole distance from P_1 to 1-10 is 20 ft., that the pole distances from P_2 to the axis, X, is 8 ft., and that the intercept, 0″-10″, on the axis, X, between the first and last sides of Polygon p_2, is 5.54, measured to scale of the elastic weights, the vertical semi-axis of the total ellipse of elasticity of the pier will be given by:

$$\rho = \sqrt{\frac{20 \times 8 \times 55.4}{74.6}} = 10.90 \text{ ft.} = B\,G_P = B'\,G_P$$

and the horizontal semi-axis by:

$$\rho' = \sqrt{\frac{18.8}{74.6}} = 0.50 \text{ ft.} = A\,G_P = A'\,G_P$$

The ellipse is then fully determined.

On Plate V, the respective positions of the two ellipses of elasticity, G_A and G_P, have been drawn to scale, also, their axes, and the top section of the pier.

Combination of Ellipses of Elasticity for Arch and Pier.—Consider the top horizontal section of the pier (Plate V), and assume that this section is common to the pier, as well as to the arch.* Any horizontal displacement, without rotation, of this section considered as a part of the pier, cannot be induced, according to the theory of the ellipse of elasticity, except by a horizontal force, R_P, applied to the pier and passing through the point, G_P. For the same reason, considering this same section as the terminal section of the arch, that same horizontal displacement must be caused by a horizontal force, R_A, applied to the arch and passing through the point, G_A, of the arch.

Considering this same section as an intermediate section of the combined system, G_P, G_A, the same horizontal displacement cannot be caused, except by the resultant of the two forces mentioned previously acting horizontally through a certain point, $G_{A,P}$, which, by the same theory, must be the center of gravity of the combination, G_A, G_P. The displacement of the section, considered as the terminal section of Pier P_1, is given by Equation (1):

$$R_P \times G_P \times \rho_P^2 = R_P \times 74.6 \times 10.90 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and when considered as a terminal of the arch, it is given by Equation (2):

$$R_A \times G_A \times \rho_A^2 = R_A \times 373.5 \times 1.30 \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

In fact, regarding this same section as an intermediate section of the combination, A_1, P_1, the displacement mentioned can be induced on it only by a horizontal force, $R_A + R_P$, passing through the center of gravity, $G_{A,P}$, of the whole system. Therefore, the vertical distances, d_1 and d_2, from the center of gravity, $G_{A,P}$, to the centers, G_A and G_P, respectively, will bear the relation:

$$\frac{d_1}{d_2} = \frac{G_P \times \rho_P^2}{G_A \times \rho_A^2}$$

and, if, $d_1 + d_2 = \delta$:

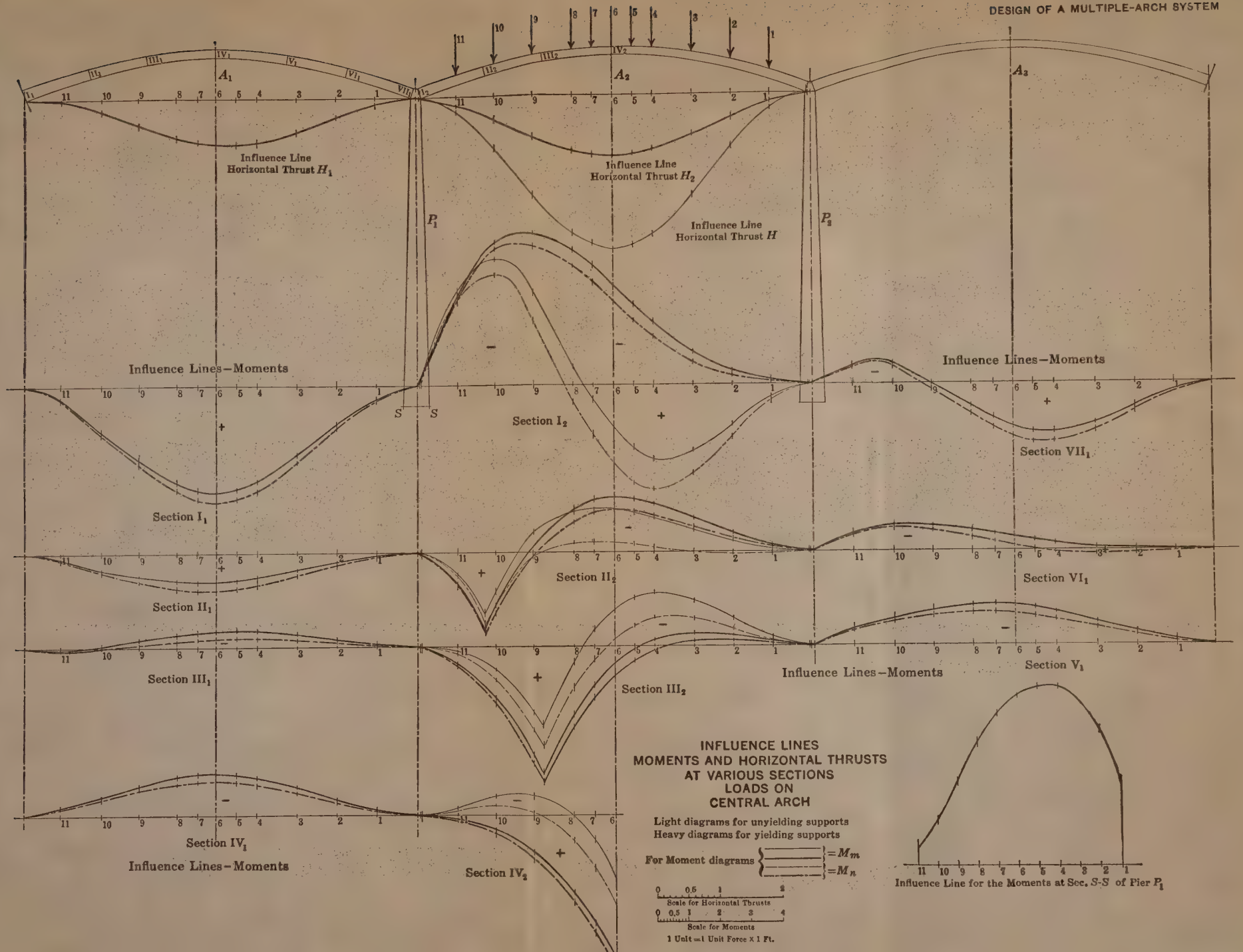
$$d_1 = \frac{1}{1 + \dfrac{G_A \times \rho_A^2}{G_P \times \rho_P^2}} \times \delta = \frac{1}{1 + \dfrac{373.5 \times 1.30^2}{74.6 \times 10.90^2}} \times 18.86 = 17.596 \text{ ft.}$$

Similarly, with the other co-ordinate of $G_{A,P}$:

$$d_1' = \frac{1}{1 + \dfrac{G_A \times \rho'^2_A}{G_P \times \rho'^2_P}} \times \delta' = \frac{1}{1 + \dfrac{373.5 \times 12.90^2}{74.6 \times 0.50^2}} \times 25.67 = 0.007 \text{ ft.}$$

in which $d'_1 + d'_2 = \delta'$.

* In considering this section as common to the pier as well as to the arch, the small parallelopiped enclosed within this section and the two end sections of the adjacent arches is disregarded. It is very small and may be ignored without material error. Otherwise, either for the sake of exactness, or because the parallelopiped is of important dimensions with respect to the voussoirs of the arch, it can be taken into consideration very easily by regarding it as an additional voussoir of the actual arch and by determining the relative quantities in the same manner as for the other voussoirs. However, when considering this parallelopiped as a voussoir, actually it is considered twice, once for each arch. In any case, however, the approximation resulting either from disregarding it, or taking it into consideration twice, is quite permissible, for the results, generally speaking, are not affected materially.


Influence Line Horizontal Thrust H_1
Influence Line Horizontal Thrust H_2
Influence Line Horizontal Thrust H
Influence Lines—Moments
Section I_1
Section I_2
Section II_1
Section II_2
Section III_1
Section III_2
Section IV_1
Section IV_2
Section VI_1
Section VII_1
Section V_1
INFLUENCE LINES
MOMENTS AND HORIZONTAL THRUSTS
AT VARIOUS SECTIONS
LOADS ON
CENTRAL ARCH
Light diagrams for unyielding supports
Heavy diagrams for yielding supports
For Moment diagrams $= M_m$ $= M_n$
Scale for Horizontal Thrusts
Scale for Moments
1 Unit = 1 Unit Force × 1 Ft.
Influence Line for the Moments at Sec. S-S of Pier P_1

The point, $G_{A,P}$, is now fully determined. It is the center of gravity of the entire elastic system, G_A, G_P, as well as the center of the ellipse of elasticity of this combined system. The position of $G_{A,P}$ being determined, the elastic weight of the combined system, $G_{A,P}$, as well as the axes of the ellipse of elasticity of this system,* is found as follows :

A rotation of the same terminal section, considered as an intermediate section of the combined system, A_1, P_1, around the point, $G_{A,P}$, cannot be induced except by a couple. Now, considering this same section as the terminal section of the pier only, the rotation must be induced by a force, R_{P_1}, acting along the anti-polar of $G_{A,P}$, with respect to the total ellipse of elasticity of the Pier P_1; furthermore, the rotation of this same section, considered as a terminal of Arch A_1, must be induced by the force, R_{A_1}, acting along the anti-polar of $G_{A,P}$, with respect to the ellipse of elasticity of the arch. Therefore, the forces, R_{A_1} and R_{P_1}, constitute a couple; that is, $R_{A_1} = -R_{P_1} = R$, and the amount of rotation is, $\phi = R \times d \times G_{A,P}$. However, considering the arch and the pier separately, the same theory of the ellipse of elasticity for the same rotation, ϕ, gives, $\phi = R \times G_A \times r_A = R \times G_P \times r_P$. Therefore :

$$G_{A,P} = \frac{G_A \times r_A}{d} = \frac{G_P \times r_P}{d}$$

that is:

$$G_{A,P} = \frac{373.54 \times 1.32}{31.80} = 15.50$$

$$G_{A,P} = \frac{74.61 \times 6.65}{31.80} = 15.60\dagger$$

Assume that, as an average, $G_{A,P} = 15.55$, which is the value of the elastic weight of the combined system, A_1, P_1.

Now, considering the horizontal displacement of the top section of Pier P_1 when this is regarded as an intermediate section of the combined system, A_1, P_1, the value of its horizontal displacement may be expressed by $(R_A + R_P)\, G_{A,P} \times \rho^2$. From Equation (1):

$$\rho^2 = \frac{G_P \times {\rho_P}^2}{G_{A,P}\left(1 + \frac{G_P \times {\rho_P}^2}{G_A \times {\rho_A}^2}\right)} = \frac{G_P \times {\rho_P}^2}{G_{A,P}\left(1 + \frac{d_1}{d_2}\right)} = \frac{74.61 \times 10.9^2}{15.55\left(1 + \frac{17.59}{1.26}\right)}$$

$$= 38.10\,;\ \rho = 6.17 \text{ ft.}$$

This equation will give the distances from the horizontal tangents of the ellipse of elasticity of the combined system, A_1, P_1, to Point $G_{A,P}$. In an identical manner, the positions of the vertical tangents to the same ellipse are determined by:

$$\rho'^2 = \frac{G_P \times {\rho'}_P^2}{G_{A,P}\left(1 + \frac{d'_1}{d'_2}\right)} = \frac{74.61 \times 0.50^2}{15.55\left(1 + \frac{0.007}{25.66}\right)} = 1.19\,;\ \rho' = 1.09 \text{ ft.}$$

Finally, as the displacement of the same section considered previously, when it is regarded as an intermediate section of the system, A_1, P_1, must

* See, also, Prof. Hool's book on this subject.

† Theoretically speaking, these two values should have been the same, but unavoidable inaccuracy of drafting gives this small and unimportant difference.

be perpendicular to a direction that is conjugate (with respect to the ellipse, $G_{A,P}$) to the line of action of the force inducing that displacement, so for this ellipse, the vertical and horizontal directions are conjugate to each other, and, therefore, the points at which these tangents will cut the axes of the ellipse will be also the ends of these axes.

Design of Arch.—There now exists a new elastic system composed of the former elastic arch with two elastic systems supporting it, one at either end, each comprising a pier and an adjacent arch, and each represented by the ellipse of elasticity just determined. It is clear, therefore, that if in designing the arch, the ellipses of elasticity of the supports are assumed as if they were the ellipses of elasticity of two ideal additional voussoirs—in other words, considering the arch as composed of twenty-two voussoirs instead of twenty, as has been done previously—the resulting stresses will be for an arch under the influence of the elasticity of its supports (Arch A_1, Pier P_1 on the left; Arch A_3, Pier P_2 on the right).

In Plate III is shown in heavy lines (for distinction) the design of this new elastic system under the assumption that the voussoirs of the half arch number eleven instead of ten as before. The method followed in determining these diagrams is identical with that for the case that assumed unyielding supports.

Influence Lines for Moments.—In Plate IV, the various diagrams (light and heavy) for the center span have been plotted. Inspection of these diagrams will reveal the great difference in the moments acting on the central arch of the system under the two assumptions.

Eleven positions have been considered for the unit load, which is supposed to move from right to left as indicated numerically. Knowing (1) the horizontal thrust for each of these positions; (2) the vertical distances between the kernel points of the various sections, and the points on the verticals passing through these kernels, in which the reactions cut these verticals, the moments acting on the various sections have been computed as shown in Table 7.

Design of Side Arch and Pier

Thus far, the influence lines of the moments have been considered only for the central arch. In order to complete this research, the influence lines for the side arches, as well as for the piers, were determined on the assumption that only the central arch was loaded.

Reactions for Side Arch and Pier.—From Plate III were obtained the reactions of the elastic supports of the arch for each loading of the central arch. These results are plotted on Plate IV, showing the stress acting on Arch A_1* and Pier P_1, while the central arch is being loaded. Each of these reactions evidently represents the combined action of the elastic system, G_A, G_P, supporting the arch and reacting against the middle arch under loading. In order to determine which part of this reaction belongs to the arch and which to the pier, the following construction was adopted.

* The diagrams for Arch A_1 hold good for the similar sections of Arch A_3.

TABLE 7.—Moments at Kernel Points for Arch A_2 (with Elastic Supports).

Position of load.	H_2.	Section I_2.				Section II_2.				Section III_2.				Section IV_2.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.132	— 1.06	— 0.60	—0.139	—0.079	—1.49	—1.09	—0.196	—0.143	—0.86	—0.50	—0.113	—0.066	+1.03	+1.36	+0.135	+0.179
2	0.381	— 1.14	— 0.69	—0.434	—0.262	—1.51	—1.11	—0.575	—0.422	—0.81	—0.44	—0.308	—0.167	+1.16	+1.49	+0.441	+0.567
3	0.677	— 1.52	— 1.04	—1.029	—0.704	—1.54	—1.14	—1.042	—0.771	—0.60	—0.23	—0.406	—0.155	+1.69	+2.02	+1.144	+1.367
4	0.913	— 2.18	— 1.69	—1.990	—1.542	—1.67	—1.26	—1.524	—1.150	—0.19	+0.16	—0.173	+0.146	+2.67	+3.00	+2.437	+2.739
5	0.983	— 2.56	— 2.07	—2.516	—2.034	—1.69	—1.28	—1.661	—1.258	+0.14	+0.51	+0.137	+0.501	+3.40	+3.78	+3.342	+3.666
6	1.009	— 3.15	— 2.67	—3.178	—2.694	—1.75	—1.31	—1.765	—1.321	+0.61	+0.99	+0.615	+0.998	+4.49	+4.82	+4.530	+4.863
7	0.983	— 3.90	— 3.37	—3.833	—3.312	—1.71	—1.28	—1.680	—1.258	+1.31	+1.69	+1.287	+1.661	.....	.....	.....	.....
8	0.913	— 4.71	— 4.18	—4.300	—3.816	—1.49	—1.04	—1.360	—0.949	+2.44	+2.83	+2.227	+2.583	.....	.....	.....	.....
	0.749	.....	.....	.....	.....	.....	.....	.....	.....	+5.42	.....	+4.059	.....	.....	.....	.....	.....
	0.751	.....	.....	.....	.....	.....	.....	.....	.....	.....	+5.75	.....	+4.318	.....	.....	.....	.....
9	0.677	— 7.19	— 6.60	—4.867	—4.468	—0.30	+0.17	—0.203	+0.115	+5.10	+5.44	+3.452	+3.682	.....	.....	.....	.....
10	0.381	—12.27	—11.50	—4.674	—4.381	+4.58	+5.16	+1.744	+1.965	+4.33	+4.67	+1.649	+1.779	.....	.....	.....	.....
	0.308	.....	.....	.....	.....	+7.86	.....	+2.420	.....	.....	.....	.....	.....	.....	.....	.....	.....
	0.315	.....	.....	.....	.....	.....	+8.20	.....	+2.588	.....	.....	.....	.....	.....	.....	.....	.....
11	0.132	—21.20	—20.77	—2.798	—2.741	+7.73	+8.09	+1.020	+1.067	+4.16	+4.50	+0.549	+0.594	.....	.....	.....	.....

A rotation, without parallel displacement of the top section of the pier considered as an intermediate section of the elastic system, G_A, G_P, when this system is under the action of a force, was performed about a center of instantaneous rotation, D. This point, according to the theory of the ellipse of elasticity is the anti-pole of the line of the acting force with respect to the ellipse of elasticity of the whole system, $G_{A,P}$. The rotation of this section considered as one end of Pier P_1, can be induced only by a certain force, the line of action of which is the anti-polar of the center, D, this time with respect to the ellipse of elasticity, G_P, of the pier. This determines the line of action of the component of the reaction due to the pier under a certain loading. Furthermore, the rotation of the same section, considered as a terminal of the arch, is induced by a force the line of action of which is the anti-polar of the same center, D, this time with respect to the ellipse of elasticity, G_A, of the arch. The line of action, therefore, of the component of the reaction due to the arch, is likewise determined. As a check on this construction, these two lines, together with the reaction, must meet in a point. Knowing the intensity of the reaction and the lines of action of the components just determined, their amount is found. Changing the signs of these two components gives the forces acting on the side arch and on the pier.

In Plate V is shown the construction required for these determinations as regards Position (4) of the load, while for other positions merely the results are indicated.

The procedure for Position (4) of the load will be considered in detail. From Plate III, the position of the left reaction, due to Load 4, as well as the envelope line and the intersection line, have been drawn to scale on Plate V. This reaction has been extended to meet the axes of the ellipse, $G_{A,P}$, as shown, and the anti-pole, D_4, with respect to the ellipse, $G_{A,P}$, has been determined.

Considering D_4 as an anti-pole with respect to the ellipse, G_P, of the pier, the polar of this anti-pole may be determined—in this case, M_4-4′. Finally, considering this same D_4 as an anti-pole with respect to the ellipse, G_A, its corresponding polar may be drawn as shown in full on Plate V; as was to be expected, it passes through Point M_4, which checks the geometrical construction. The two lines so determined will represent the reactions due to the pier and to the arch, respectively.

As in the previous case, having determined the amount and direction of the reaction on the arch, A_1, its horizontal and vertical components may be obtained. The horizontal component will represent the horizontal thrust of the arch, and the vertical component will represent the vertical reaction of the left support of Arch A_1. This operation being performed for every force acting on the arch gave the horizontal thrust acting on the arch, as well as the left reaction of Arch A_1, for each position of the unit load acting on Arch A_2. These results are shown on Plate V.

The horizontal thrust on Arch A_1 being known for each position of the load on Arch A_2, the relative moments acting on the sections of Arch A_1 were computed, by multiplying, in each instance, the horizontal thrust by the distance between the kernel points of the sections under consideration and the line of action of the force; these distances were measured on the verticals

TABLE 8.—MOMENTS AT KERNEL POINTS FOR ARCH A_1 UNDER LOADS ON ARCH A_2.

Position of load.	H_1.	Section I_1.				Section II_1.				Section III_1.				Section IV_1.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.115	+ 3.94	+ 4.36	+0.453	+0.501	+ 0.91	+1.27	+0.104	+0.146	−0.79	−0.44	−0.090	−0.050	− 1.60	− 1.27	−0.184	−0.146
2	0.322	+ 3.97	+ 4.38	+1.278	+1.410	+0.94	+1.29	+0.302	+0.415	−0.77	−0.42	−0.247	−0.135	− 1.62	− 1.29	−0.521	−0.415
3	0.555	+ 4.03	+ 4.45	+2.236	+2.469	+ 0.97	+1.33	+0.538	+0.738	−0.75	−0.40	−0.416	−0.222	− 1.64	− 1.31	−0.910	−0.727
4	0.735	+ 4.19	+ 4.59	+3.079	+3.373	+1.05	+1.42	+0.771	+1.043	−0.73	−0.38	−0.536	−0.279	− 1.67	− 1.34	−1.227	−0.984
5	0.775	+ 4.27	+ 4.69	+3.309	+3.634	+1.11	+1.48	+0.860	+1.147	−0.69	−0.34	−0.534	−0.263	− 1.69	− 1.36	−1.309	−1.054
6	0.770	+ 4.42	+ 4.83	+3.403	+3.719	+1.19	+1.56	+0.916	+1.201	−0.67	−0.31	−0.515	−0.238	− 1.75	− 1.42	−1.347	−1.093
7	0.730	+ 4.59	+ 5.00	+3.350	+3.650	+1.30	+1.67	+0.949	+1.219	−0.65	−0.29	−0.474	− 0.211	− 1.79	− 1.46	−1.306	−1.065
8	0.652	+ 4.77	+ 5.16	+3.110	+3.364	+1.38	+1.74	+0.899	+1.134	−0.62	−0.27	−0.404	−0.176	− 1.86	− 1.53	−1.212	− 0.997
9	0.435	+ 5.36	+ 5.77	+2.331	+2.509	+1.69	+ 2.04	+0.735	+0.887	−0.58	−0.22	−0.252	− 0.095	− 2.11	− 1.78	−0.917	−0.774
10	0.185	+ 7.51	+ 7.90	+ 1.389	+1.461	+ 2.84	+ 3.18	+0.525	+0.588	−0.30	+0.04	− 0.055	+0.007	− 2.88	− 2.55	−0.532	−0.471
11	0.020	+17.84*	+17.94†	+0.356	+0.358	+ 9.00*	+9.11†	+0.180	+0.182	+3.35	+3.62	+ 0.067	+0.072	−12.23	−11.90	−0.244	−0.238

TABLE 8.—(*Continued*).

Position of load.	H_1.	Section V_1.				Section VI_1.				Section VII_1.			
		h_m.	h_n.	M_m.	M_n.	h_m.	h_n.	M_m	M_n.	h_m.	h_n.	M_m.	M_n.
1	0.115	− 1.26	− 0.91	−0.144	−0.104	0	+ 0.36	0	+0.041	+ 2.52	+ 2.94	+0.289	+0.338
2	0.322	− 1.28	− 0.93	−0.412	−0.299	− 0.03	+ 0.33	−0.009	+0.106	+ 2.48	+ 2.90	+0.798	+0.933
3	0.555	− 1.33	− 0.98	−0.738	−0.543	− 0.10	+ 0.26	−0.055	+0.144	+ 2.37	+ 2.79	+1.315	+1.548
4	0.735	− 1.44	− 1.08	−1.058	−0.793	− 0.28	+ 0.09	−0.205	+0.066	+ 2.14	+ 2.56	+1.572	+1.881
5	0.775	− 1.50	− 1.14	−1.162	−0.883	− 0.37	− 0.01	−0.286	−0.007	+ 2.02	+ 2.44	+1.565	+1.891
6	0.770	− 1.63	− 1.27	−1.255	−0.977	− 0.55	− 0.18	−0.423	−0.138	+ 1.77	+ 2.19	+1.362	+1.686
7	0.730	− 1.76	− 1.41	−1.284	−1.029	− 0.75	− 0.38	−0.547	−0.277	+ 1.49	+ 1.92	+1.087	+1.401
8	0.652	− 1.93	− 1.57	−1.258	−1.023	− 0.99	− 0.62	−0.645	−0.404	+ 1.15	+ 1.58	+0.749	+1.030
9	0.435	− 2.48	− 2.12	−1.078	−0.922	− 1.80	− 1.44	−0.783	−0.626	+ 0.06	+ 0.50	+0.026	+0.217
10	0.185	− 4.27	− 3.91	−0.789	−0.723	− 4.47	− 4.08	−0.826	−0.754	− 3.58	− 3.14	−0.662	−0.580
11	0.020	−26.65	− 26.19	−0.533	−0.523	−19.54*	−19.29†	−0.390	−0.385	−25.48*	−25.12†	−0.509	−0.502

* Normal distance between m and line of action of "11".

† Normal distance between n and line of action of "11".

passing through the kernel points of the section under investigation. These computations were made for Sections I_1 to VII_1, inclusive (see Table 8), and the corresponding influence lines of the moments were plotted on Plate IV.

The vertical reaction of the support, P_1, has also been plotted; this is merely Polygon p_3 of Plate III developed and drawn to scale. By repeating for Pier P_1 the operations already described for Arch A_1, the horizontal components of the forces were computed and the moments acting on the pier found. These moments are shown on Plate IV. Inspection of Plate IV, shows at a glance that there is a remarkable difference in the results obtained by the two different hypotheses.

Yielding of Soil.—It should be noted that, in this problem, the yielding of the soil has not been considered, because of the light form of construction contemplated and of the possibility of stabilizing the foundation even if the soil were somewhat yielding.

However, in case of peculiar conditions such as a somewhat yielding soil and no means of making it rigid, the elasticity of the foundation should be considered in designing an arch. This does not complicate the problem in any way; it only means the addition of an ellipse, due to the yielding of the soil,* to the various ellipses of elasticity of the pier sections. The designer should investigate the behavior of a multiple-arch system in every case where the importance of the spans, the comparative slenderness of the piers, and the conditions of the foundation are such as to warrant a thorough investigation.

Dead Loads.—The polygon of pressure due to dead load, which, in this case, is the same for all three arches, as all are equal and equally loaded, is not affected by the two hypotheses under which Arch A_2 and, consequently, Arches A_1 and A_3 were designed. Therefore, it will not be discussed here.

Temperature.—The effect of temperature change, as well as that of shrinkage in concrete due both to the setting of the concrete and to the dead and live loads, is not presented in this paper. The problem has been solved only for the live load, and in a manner, which, in the present state of knowledge, represents the most complete solution that can be desired.

Conclusions

From the results obtained in this investigation, as represented by the diagrams of Plate IV, the following conclusions seem to be justified.

The conditions assume an elastic system composed of several arches, supported by comparatively slender piers, each arch being regarded as having fixed ends. According to theory, this elastic system should be considered as a unit by itself, and the design carried out accordingly. The analytical methods that should be followed, in this instance, would be exceedingly cumbersome and laborious, and the graphical method, which has been applied in the case of three arches, although very simple and elegant, would become rather laborious in application if extended to several arches.

The question arises, therefore, whether it is possible, for practical purposes, to simplify or shorten this task of the designer in a case like that mentioned. Professor Guidi, of the Polytechnic of Turin, Italy, has suggested a

* As explained in Prof. Hool's book, previously mentioned.

simplification which would eliminate a great deal of work for the designer. He would consider three adjacent arches of the system as forming an elastic system by itself, disregarding what is on either side of this selected unit, that is, the outer ends of this three-arch system would be regarded as fixed. This assumption, however, would only be strictly correct if all the members of the system were of the same dimensions.

Considering the influence lines of Plate IV, the statement is justified that if the arches of this system, no matter how many, are reinforced in such a way as to withstand the conditions of loading, as shown in this paper, the system is to be regarded as safely designed as far as live load is concerned; and if the piers are reinforced to withstand the condition of loading, as shown for Arch A_2, they are safe.

APPENDIX

Graphical Treatment of Statical Moments, Products of Inertia, Moment of Inertia, and Ellipse of Inertia, Leading to the Conception and Construction of the Ellipse of Elasticity. Method to Be Followed in Applying this Theory to a Plane Elastic System*

General Principles

1.—Statical Moment.—Let F_1, F_2, and F_3 (Fig. 2) be three parallel coplanar forces. To obtain the statical moments of these three forces with respect to a given axis, x, construct the equilibrium polygon on F_1, F_2, and F_3, with respect to the polygon of forces, 0, 1, 2, and 3, and the pole, P. The segments, $0'1'$, $1'2'$, and $2'3'$, are proportional to the "statical moments" desired.

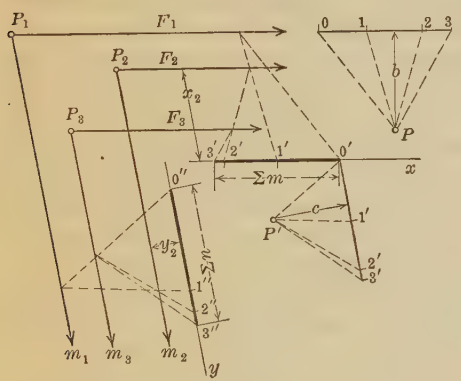


FIG. 2.

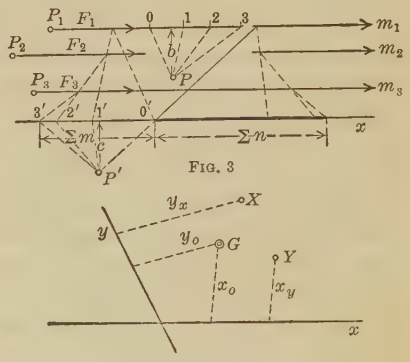


FIG. 3

FIG. 4.

2.—Product of Inertia.—Let y be another given axis in the same plane. Through the points of application, P_1, P_2, and P_3, of the forces F_1, F_2, and F_3,

* The theory of the ellipse of elasticity was first announced by Culmann in his "Die Graphische Statik" (Zurich, 1875). Later, W. Ritter, his successor in the Polytechnic in Zurich, Switzerland, used this principle continuously, especially in his works, "Der Elastische Bogen" (Zurich, 1886), and, "Anwendungen der Graphischen Statik" (Zurich, 1888–1906).

For the generation of this ellipse, Ritter uses an entirely graphical method, which is far simpler and quicker than analytical methods, as, for instance, that used by Dr. Postuvanschitz, mentioned later. For further information concerning the geometrical principles governing the method used by Ritter in its demonstration, see "Elements of Projective Geometry," by L. Cremona, translated by Charles Leudesdorf, 1885, Clarendon Press, Oxford.

Since then, this theory has been extensively taught and used in Europe, especially for the solution of the most difficult problems of engineering, where the analytical methods are found impractical.

Besides the books mentioned, there are others worthy of note, dealing with this subject, among which, for instance, may be mentioned: "Teoria dell'Elasticita e Resistenza dei Materiali," Parte Seconda, and "L'Arco Elastico senza Cerniere," by Camillo Guidi, 1903; "Contributo alla Trattazione Grafica dell'arco Continuo su Appoggi Elastici," by M. Panetti, 1901; and "Fortschritte der Ingenieurwissenschaften," by Dr. F. Postuvanschitz; the author of this book deals with this subject mainly by analysis; Prof. H. Lossier, in 1913, also published in *Genie Civil* a paper dealing with the application of this method to a system of arches.

In May, 1913, the writer presented a paper before the Western Society of Engineers, *Proceedings*, Vol. XVIII, No. 5, dealing with this subject, under the title, "Designing of an Arch System," which work seems to have been the first of its kind written in the English language. In the third volume of "Reinforced Concrete Construction," by Prof. G. A. Hool, there is a treatment of this graphical method. In April, 1917, S. Moreell, Jr., also presented a paper before the Western Society of Engineers dealing with this subject. A learned criticism of this theory is given by George F. Swain, Past-President, Am. Soc. C. E., in a paper entitled "On a New Principle in the Theory of Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 702 *et seq.*

draw lines parallel to y. Assume forces of the same magnitude as $0'1'$, $1'2'$, and $2'3'$, the statical moments with respect to the axis, x, to lie, respectively, in these lines parallel to y, and construct on them the equilibrium polygon the pole of which is P' (Fig. 2), giving the segments, $0''1''$, $1''2''$ and $2''3''$, on the line, y, which are proportional to the products of inertia of the forces, F_1, F_2, and F_3, with respect to the axes, x and y. The following equation gives for any one of the forces, say, F_2, the double moment called the "product of inertia", with respect to the axes, x and y:

$$\text{Product of Inertia of } F_2 = F_2 \cdot x_2 \cdot y_2 = b \cdot c \cdot 1''2''$$

in which, b and c are the polar distances. Also, for the whole system, $\Sigma F x y = b \cdot c \cdot \Sigma n$ (Fig. 2).

3.—Moment of Inertia.—If the axis, y, coincides with the axis, x, the product of inertia assumes the special value called the "moment of inertia".

Let F_1, F_2, and F_3 be three parallel co-planar forces. To obtain the moment of inertia with respect to a given axis, x, a procedure analogous to that of Paragraph 2 is followed (Fig. 3). The moment of inertia of the whole system $= \Sigma F x^2 = b \cdot c \cdot \Sigma n$.

The expression, $\Sigma F x^2 : \Sigma F$, is of the second degree; hence, it can be represented by the square of a length, r; this is what is called the "radius of inertia" (radius of gyration), therefore:

$$r^2 = \frac{\Sigma F x^2}{\Sigma F}$$

Let x be an axis lying in the plane, and x_0, an axis parallel to x, passing through the center of gravity of the system; d, the distance between these two axes; and x and x_0, the distances from P, a point of application of one of the forces of the system to these axes (these distances, d, x, and x_0 are taken in an arbitrary direction); $x = x_0 \pm d$, and, therefore, knowing that $\Sigma F x_0 = 0$,

$$\Sigma F x^2 = \Sigma F x_0^2 + d^2 \cdot \Sigma F \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

that is:

The moment of inertia of a system with respect to any axis is equal to the moment of inertia of this system with respect to its gravity axis parallel to the given axis plus the product of the summation of the forces and the square of the distance between the two axes.

If r and ρ are the radii of inertia of the system with respect to Axes x and x_0, then $r^2 = \rho^2 + d^2$, that is:

The square of the radius of inertia with respect to any axis is equal to the square of the radius of inertia with respect to the gravity axis parallel to the given axis plus the square of the distance between the two axes.

4.—Center of Gravity of Static Moments.—Any system of forces, such as F_1, F_2, and F_3, has a center of gravity; so also, considering their statical moments about any axis in their plane as a new system of forces, the latter must have a center of gravity. This is called the center of gravity of the statical moments, or briefly, "the center relative to that axis". Its position depends on that of the axis of moments. To locate this center relative to the axis, lay off lines representing to scale the magnitudes of the static moments considered as forces, and proceed precisely as in finding the center of gravity of a plane system of forces. Subsequently, several important properties of such centers are given.

5.—Conjugate Axes.—In Fig. 4, let G be the center of gravity of a system of forces, X, the center relative to the axis, x, and Y, the center relative to the axis, y (that is, X is the center of gravity of the statical moments of the forces about the axis, x; Y, that about y).

Also, let the distances from G, X, and Y, to the axes be computed in any assumed direction; then, since the sum of the statical moments of the forces of the system with respect to the axis, x, equals the moment of the resultant with respect to the same axis:

$$\Sigma F x = x_0 . \Sigma F$$

Then, as explained previously, if the individual products, $F . x$, are regarded as forces, and their static moments with respect to y found, the following equation may be written:

$$\Sigma (F x) y = \Sigma F x y = y_x . \Sigma F x$$

Substituting, further, the value for $\Sigma F x$:

$$\Sigma F x y = y_x . x_0 . \Sigma F$$

If the order of computing the statical moments is reversed:

$$\Sigma F y x = x_y . y_0 . \Sigma F$$

Combining the last two equations:

$$y_x . x_0 = x_y . y_0$$

If neither x nor y passes through G, that is, neither x_0 nor y_0 is zero, then, to satisfy this equation, if y_x is zero, x_y will also be equal to zero. Hence:

If the center relative to an axis falls on another axis, the center relative to the latter will fall on the former.

Again, if an axis, y, passes through a center, X, its relative center, Y, will fall on the axis, x, of which X is the relative center. Therefore, any axis passing through the given point, X, the relative center of the axis, x, will have its relative center lying on x. Hence, to each line of the set of lines in the plane considered as passing through X (pencil of rays in the language of projective geometry), will correspond a definite point of a set of points (range) on the axis, x, of which X is the relative center. There is, therefore, a "projective correspondence" between axis and center. Two axes, situated so that each passes through the center (pole) of the other, are called "conjugate axes".

In view of the preceding, either of the equations:

$$\Sigma F x y = y_x . x_0 . \Sigma F$$

or:

$$\Sigma F y x = x_y . y_0 . \Sigma F$$

may be thus stated:

The product of inertia of a plane system of forces with respect to two conjugate axes is zero, and conversely.

6.—Applying this reasoning to the special case of the product of inertia that is called the moment of inertia, the latter may be found with respect to an axis, x, and, calling x_x the distance from the center, X, to the axis, x, Equation (4) is derived:

$$\Sigma F x^2 = x_x . x_0 . \Sigma F \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

or,

$$x_x = \frac{\Sigma F x^2}{x_0 \cdot \Sigma F}$$

The value of x_x, if the axis, x, passes through the center of gravity, G (when $x_0 = 0$) is ∞ which means that:

The center relative to an axis passing through the center of gravity falls on the line lying at an infinite distance away.

In this case, to the pencil having its center in the center of gravity, G, will correspond a range of points lying on the line at ∞, so that it may be said that this line at ∞ is the axis of the center of gravity of the system.

7.—Let G, x, X (Fig. 5), be, respectively, the center of gravity of the system, an axis, and its corresponding center. Take any point, Y, on this axis with y its corresponding axis, which will pass through X; the meeting point, Z, of this axis and the axis, x, will have its corresponding axis, z, passing through X and Y. The points, X, Y, and Z, of this triangle, together with the straight lines, x, y, and z, constitute three pairs of elements which correspond to each other in a double manner; that is, to the center, X, corresponds the line, x, and to the line, x, corresponds the center, X, and, similarly, with the two remaining pairs. The center, Y, and its corresponding axis, y, however, have been chosen arbitrarily, so it may be concluded that the whole system of centers and axes lying on this plane constitutes a polar system, the center of which falls on the center of gravity of this system.

Point X' (the intersection of $G X$ with the axis, x) is the center corresponding to the axis through X, parallel to x; X and X' are conjugate centers. If all the forces have the same direction, the distance between a center (pole) and its corresponding axis (polar) is always greater than the distance between the center of gravity of this system and its polar; it follows that, in this case (the only one which is of engineering interest), the auxiliary conic of the system is imaginary. Let X_1 be the symmetrical point, X, with respect to G and imagine the same construction for all the points like X, Y, Z, of the system. Considering all points like X_1 as poles and all axes like x as their corresponding axes, there results a new polar system, the auxiliary conic of which is real and is an ellipse; it is called the "central ellipse of inertia".

Any axis and its corresponding center are polar and anti-pole (that is, symmetrical point of the pole with respect to the center of gravity) with respect to the central ellipse of inertia (auxiliary conic of the polar system). The center (pole) corresponding to a tangent (axis) to this ellipse is the point which is symmetrical to the point of contact of this ellipse. Let x be an axis tangent to the ellipse, and let ρ be the length of the half-diameter conjugate with the direction of x. Taking the moment of inertia, $\Sigma F x^2$, with respect to the axis, x, and computing the distances in the conjugate direction, $x_x = 2\rho$, $x_0 = \rho$, and, therefore, on account of Equation (4):

$$\Sigma F x^2 = 2\rho \cdot \rho \cdot \Sigma F$$

and, therefore, for Equation (3):

$$2\rho \cdot \rho \cdot \Sigma F = \Sigma F x_0^2 + \rho^2 \cdot \Sigma F$$

from which:

$$\rho^2 = \frac{\Sigma F x_0^2}{\Sigma F}$$

that is:

The radius of inertia (gyration) corresponding to any axis passing through the center of gravity is given by that half diameter of the ellipse of inertia, the direction of which is conjugate with that of the assumed axis.

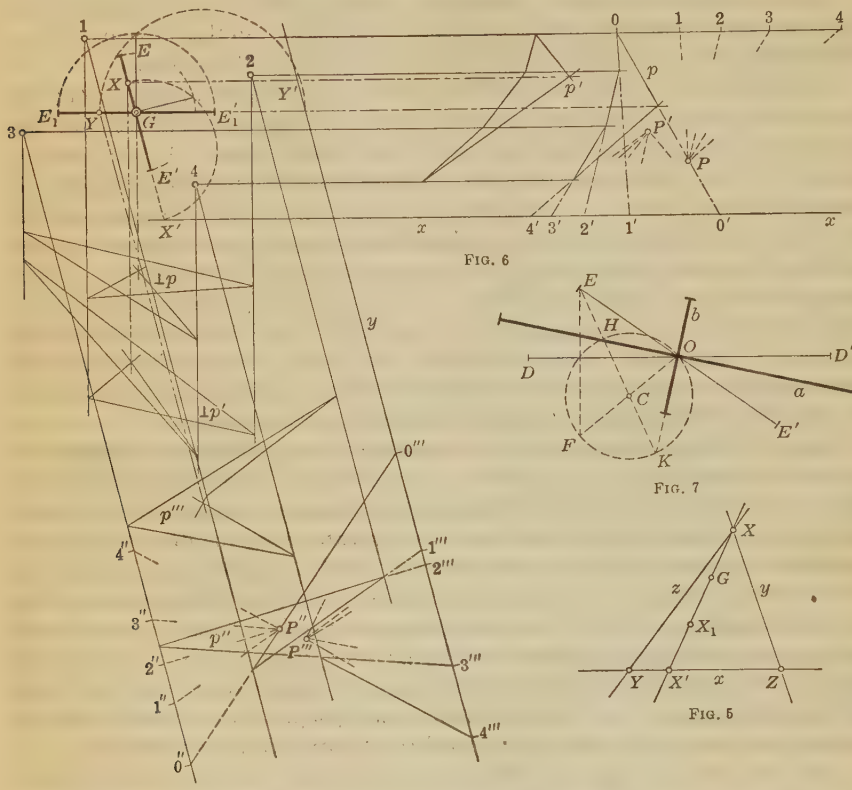


FIG. 6

FIG. 7

FIG. 5

Therefore, the central ellipse of inertia is simply the polar diagram of the law of variation according to which the moment of inertia, with respect to an axis, varies while this axis rotates about the center of gravity of the system. The moment of inertia is maximum and minimum when the axes of the ellipse are, respectively, the minor and major axes; it is for this reason that these two axes are called the "principal axes of inertia" passing through the center of gravity.

8.—Construction of the Central Ellipse of Inertia for a System of Concentrated Forces.—Let 1, 2, 3, and 4 (Fig. 6), be the points of application of four parallel forces and let x be an arbitrary axis. Construct the force line, 0, 1, 2, 3, 4, parallel to the axis, x, with P as a pole. After extending these forces in a direction parallel to the axis, x, connect them by the equilibrium

(funicular) polygon, p, which will give the measure of the static moments, $0'\,1'$, $1'\,2'$, $2'\,3'$, $3'\,4'$, and also the line of action of the resultant force through the center of gravity, G, of the system. Take another pole, P', and connect the static moments just mentioned as new forces applied to the same points, 1, 2, 3, 4, in the same direction as the axis, x. Construct the equilibrium polygon, p', corresponding to this assumption and the resultant line of these static moment-forces will contain X, the anti-pole corresponding to the axis, x. In order to determine the positions of G and X, rotate the given forces through an angle of 90°, also, the static moment-forces, and repeat this same graphical construction. For a rotation of 90°, the two equilibrium polygons will have their sides at 90° with the corresponding sides of the two polygons, p, and p'; it is, therefore, not necessary to construct force polygons, P and P', in this new direction. The lines of resultants given by these two new equilibrium polygons, $\perp p$ and $\perp p'$, will meet the lines of the resultants of the first system at the points, G and X, required. Draw the line, $G\,X$; the points, X and X', are conjugate, and, therefore,

$$\sqrt{G\,X \times G\,X'} = G\,E$$

which is the length of the semi-diameter of the ellipse of inertia in the direction shown.

Let y be an axis parallel to $G\,X$; the anti-pole, Y, must fall on the axis passing through G in a direction parallel to x. Lay off $0''$, $1''$, $2''$, $3''$, $4''$, as a line of forces parallel to the axis, y, with P'' as a pole; considering the forces in a direction parallel to y and connecting them with an equilibrium polygon, p'', there is determined on the axis, y, the intercepts, $0''\,'$, $1''\,'$, $2''\,'$, $3''\,'$, $4''\,'$, which are proportional to the statical moments of the given forces with respect to this axis. Now, choose any pole, $P''\,'$, and construct another equilibrium polygon, $p''\,'$, with respect to the intercepts and the pole just mentioned. The line of resultant thus obtained will meet the axis passing through G and parallel to x at the point, Y. The centers, Y and Y', are conjugate and, therefore,

$$\sqrt{G\,Y \times G\,Y'} = G\,E_1,$$

which is the semi-diameter of the ellipse that is conjugate to the previously found semi-diameter. The lengths and positions of its two conjugate semi-diameters being known, the ellipse is fully determined.

9.—It may be useful to know a method whereby it is possible to determine the axis of an ellipse without drawing the ellipse itself. If $D\,D'$ and $E\,E'$ (Fig. 7) are two conjugate diameters of an ellipse, from E draw $E\,F$ perpendicular to $D\,D'$ so that $E\,F = O\,D$. With C the middle point of $F\,O$ as a center, pass a circle through F and O. Join C with E, determining the points, H and K; $O\,H$ and $O\,K$ are the positions of the axes of the ellipse—the half lengths will be $E\,K$ and $E\,H$.*

10.—Given the Polar of an Ellipse to Find Its Anti-Pole, and Given the Anti-Pole to Find Its Polar.—Let p, Fig. 8, be a polar with respect to an ellipse the axes of which are $A\,B$ and $C\,D$. Lay off on $O\,C$ the segment, $O\,A' = O\,A$;

* Chasles, "Section Coniques," Paris, 1865.

similarly, the segment, $O\,C' = O\,C$. Join E with A' and F with C' (D and E being the intersections of p with the y and x-axes, respectively); then, through A' draw $A'\,P_x$ normal to $E\,A'$, and, similarly, through C', obtaining $C'\,P_y$.

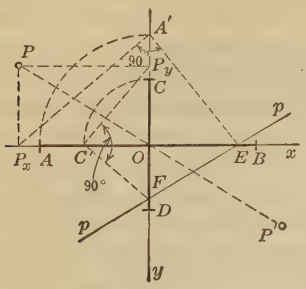


FIG. 8.

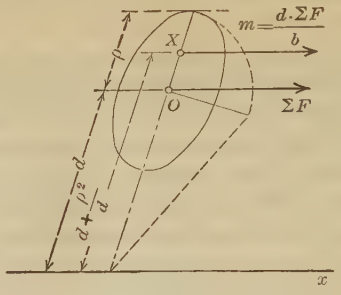


FIG. 9.

The point, P, having $O\,P_x$ and $O\,P_y$ as its co-ordinates, will be the anti-pole of p with respect to the given ellipse. The pole of p will be the point, P', which is symmetrical with P in respect to the center, O.

11.—To find the moment of inertia of a system of co-planar forces with respect to an axis, or the anti-pole of an axis, which are necessary for the construction of the conic of inertia of the system, assume that the given forces are divided into groups. In order to determine the static moments of these forces with respect to the axis, x, substitute the resultant of each one of these groups of forces for the several forces constituting each group, and assume this resultant to be applied to the center of gravity of the group. In fact, if ΣF is the summation of the forces of any one group of the system, and if d is the distance of the center of gravity of this group from the axis, x, $\Sigma F x = d \,.\, \Sigma F$. In order to determine the moments of inertia with respect to the same axis, x, or else to determine the position of the anti-pole, X, of the whole system with respect to the axis, x, it is possible to substitute the static moment-forces of the resultants of the various groups for the static moments of the single forces, provided, however, that instead of applying these static moments to the center of gravity of each group, they are applied to the centers of gravity of the static moment-forces of each group, that is, to the anti-poles of the axis, x, with respect to the central ellipses of the various groups (see Fig. 9). The distance between one of these anti-poles and the axis, x, is $d + \frac{\rho^2}{d}$, in which, ρ is the radius of inertia with respect to an axis passing through the center of gravity of that group and parallel to the axis, x. The moment of inertia of that group, therefore, is expressed by:

$$\Sigma\, F\, x^2 = d \,.\, \left(d + \frac{\rho^2}{d}\right) \,.\, \Sigma\, F$$

Two equilibrium polygons as described in the preceding paragraphs will give graphically the product, $d \,.\, \Sigma F = b \,.\, m$, as well as the other product, $m \left(d + \frac{\rho^2}{d}\right) = c \,.\, n$. For each group of forces, therefore, the line of action

of the resultant will act, in the case of the first polygon at a distance, d, from the axis and be applied to the center of gravity of the group; for the second polygon it will act at a distance, $d + \frac{\rho^2}{d}$, from this axis and be applied to the anti-pole of its axis with respect to the ellipse of inertia of that group. This shifting of the line of action is called the "displacement of forces". When for a given group of forces the distance, d, is very large as compared with the radius of inertia, ρ, the displacement, $\frac{\rho^2}{d}$, becomes so small that it may be neglected.

It has been shown how the conception and the generation of the ellipse of inertia of a system of forces by means of graphical methods is possible. In a similar way, the ellipse of inertia of an area may be constructed by substituting for the force or group of forces, an area. The generation of the ellipse of elasticity, therefore, which is nothing other than the ellipse of inertia of some quantities called elastic weights, should by geometrical analogy be already clear.

Before entering into any direct explanation of the theory of the ellipse of elasticity, it is thought expedient to state the following two theorems.

12.—Composition of Rotations.—Let P, Fig. 10, be a point of an elastic system, which, by any means, is moved an infinitesimal distance to P'. The displacement, PP', may be regarded as a rotation about an instantaneous center, say O_1, through an angle, ϕ_1. Let there be another displacement starting from the same point, P, through an angle, ϕ_2, about a center, O_2.

Join O_1 and O_2 by a straight line and erect $A\,P = y$ perpendicular to $O_1\,O_2$. Decompose $P\,P'$ into $P\text{-}P_{y_1}$, perpendicular, and $P\text{-}P_{x_1}$, parallel, to $O_1\,O_2$. Then, the triangles, $A\,P\,O_1$ and $P\,P'\,P_{x_1}$, are similar, as their sides are perpendicular. Also, since the angles of rotation are very small, $\tan . \phi = \phi$. Hence,

$$\frac{P\,P'}{P\,P_{x1}} = \frac{P\,O_1}{P\,A}$$

but,

$$P\,P' = P\,O_1 . \tan \phi_1 = P\,O_1 . \phi_1, \text{ and } \frac{P\,O_1}{P\,P_{x1}}\,\phi_1 = \frac{P\,O_1}{y}$$

Therefore,

$$P\,P_{x1} = \phi_1 . y$$

Also,

$$\frac{P\,P'}{P\,P_{y1}} = \frac{P\,O_1}{A\,O_1}, \quad \frac{\phi_1}{P\,P_{y1}} = \frac{1}{x_1}$$

Therefore,

$$P\,P_{y1} = \phi_1 . x_1$$

Similarly, for a rotation about O_2 from P:

$$P\,P_{x2} = \phi_2 . y$$

$$P\,P_{y2} = \phi_2 . x_2$$

The sums of the components of these displacements will be:

$$P P_y = P P_{y1} + P P_{y2} = \phi_1 \cdot x_1 + \phi_2 \cdot x_2$$
$$P P_x = P P_{x1} + P P_{x2} = \phi_1 \cdot y + \phi_2 \cdot y = (\phi_1 + \phi_2)\, y$$

Let O be the center of gravity of forces proportional to ϕ_1 and ϕ_2 applied, respectively, at O_1 and O_2. If the point, P, rotates about the center, O, through an angle, $\phi = \phi_1 + \phi_2$, the components of this displacement, in the same directions as previously stated, will be given, as proved for the first rotation, by:

$$P_y = \phi \cdot x = \phi_1 \cdot x_1 + \phi_2 \cdot x_2$$
$$P_x = \phi \cdot y = (\phi_1 + \phi_2)\, y$$

These values are identical with those already found for the sums of the components of the two displacements, therefore: The resultant rotation of two rotations of a point, P, about the centers, O_1 and O_2, respectively, through the angles, ϕ_1 and ϕ_2, is a rotation through an angle, $\phi = \phi_1 + \phi_2$, about the center of gravity, O, of forces proportional to ϕ_1 and ϕ_2, applied, respectively, at the centers, O_1 and O_2.

13.—Principle of Reciprocity.—The theorem of reciprocity by Betti and its corollary by Maxwell may be demonstrated graphically as follows: Let P_1 and P_2 in Fig. 11 be two points belonging to the same plane elastic system. A force, F_1, rigidly connected with P_1, acts on the system, increasing from zero to its maximum value, F_1.

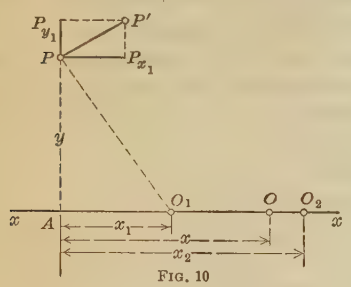


FIG. 10

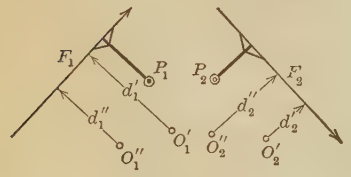


FIG. 11.

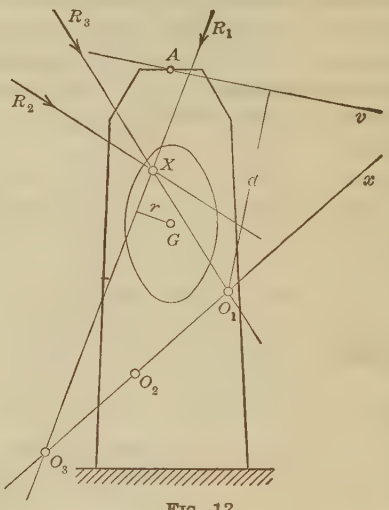


FIG. 12.

The effect on P_1 is to rotate the latter through an angle, ϕ_1' about an instantaneous center of rotation, O_1'. When F_1 reaches its maximum value, a second force, F_2, rigidly connected with P_2, begins to act with an initial value, zero, increasing to its maximum, F_2. A corresponding rotation through an angle, ϕ_2'', about some center, O_2'', results.

The rotation of P_1 caused by F_1 (constant at the value, F_1, and still acting as described), during this interval, will be about some center, O_1'', and will extend through some angle, ϕ_1''. When F_2 has reached its final value, F_2, the virtual work done by both forces on the system, will be:

$$\frac{1}{2} F_1 \cdot \phi_1' \cdot d_1' + \frac{1}{2} F_2 \cdot \phi_2'' \cdot d_2'' + F_1 \cdot \phi_1'' \cdot d_1''$$

If the order of action of F_1 and F_2 on the system is reversed, and if the angle of rotation of P_2 about the new center, O_2', during the action of F_1 is ϕ_2', the virtual work will be:

$$\frac{1}{2} F_2 \cdot \phi_2'' \cdot d_2'' + \frac{1}{2} F_1 \cdot \phi_1' \cdot d_1' + F_2 \cdot \phi_2' \cdot d_2'$$

As, however, the order of action of these two forces is immaterial, as far as the final and initial positions of the elastic system are concerned, the two expressions for the virtual work may be equated, and after cancellation, there finally results:

$$F_1 \cdot \phi_1'' \cdot d_1'' = F_2 \cdot \phi_2' \cdot d_2'$$

which means that:

The virtual work of the force, F_1, during the action of the force, F_2, is the same as the virtual work of the force, F_2, during the action of F_1.

14.—Ellipse of Elasticity.—Let A (Fig. 12) be a terminal point of an elastic body, and R_1 a force rigidly connected to Point A and acting on the whole system. The displacement of Point A caused by the force, R_1, may be regarded as a rotation around a certain point, O_1 (center of instantaneous rotation). If there is another force, R_2, acting on the system similar to R_1, then the displacement of Point A caused by this second force may be regarded as a rotation around another point, O_2 (center of instantaneous rotation). From what has been stated previously, the resultant of R_1 and R_2 will cause a displacement of A the center of instantaneous rotation of which is a point, O_3, lying on the line joining Points O_1 and O_2; that is, if a force rotates about a point, X, the corresponding center of instantaneous rotation moves on a straight line, x.

If O_3 is the center relative to the force, R_3, passing through O_1, then, since during the action of the force, R_1, the force, R_3, does not do any work, by the principle of reciprocity, during the action of the force, R_3, the force, R_1, cannot do any work; therefore, O_3 will fall on R_1, that is:

If a force passes through the center of instantaneous rotation relative to another force, this last force passes through the center of instantaneous rotation relative to the first force.

If a force acts along the straight line, x, its relative center of rotation will fall on X, for since this force passes through O_1, O_2, its relative center of rotation, must fall simultaneously on R_1 and R_3. There are, therefore, three points, X, O_1, and O_3, and three straight lines, x, R_1, and R_3, forming three pairs of elements which correspond to each other in a double manner, namely, to the center, X, corresponds the straight line, x, and to the straight line, x,

corresponds the center, X, and, similarly, for the two remaining pairs. These geometrical relations are true for all straight lines and all points of the plane, and, therefore, they constitute a polar system. The auxiliary conic of this system is imaginary, but considering these lines and all points symmetrical to the centers of rotation with respect to the center of gravity, then there is another polar system, the auxiliary conic of which is real and is an ellipse. With respect to this ellipse, any force and its instantaneous center of rotation are to be considered as polar and anti-pole.

It is this ellipse which is called the "ellipse of elasticity" of a given elastic system, or the ellipse of elasticity of its terminal point, A.

In Fig. 13, G represents the center of gravity of a certain polar system; c represents the auxiliary conic of this polar system; and X_1 and x represent a pair of the infinite number of conjugate pairs of elements of this polar system (X_1 being the pole and x being the polar). It can be seen that the polar passes through the contact points of the two tangents to the conic drawn from Point X_1. X is the anti-pole of x corresponding to Pole X_1 and is taken so that $G X_1 = G X$.

If a force acts along the line, x, and on the terminal point of an elastic system, and X is the center of instantaneous rotation of this terminal section due to the action of this force, and assuming an infinite number of these lines of action of forces acting on the same terminal section of the system, there will be a corresponding infinite number of centers of instantaneous rotation. All these pairs of elements, like x and X, constitute a polar system, which, for projective reasons, admits only of an imaginary auxiliary conic. If, however, for the infinite points like X, is substituted its symmetrical, X_1, with respect to G, the new polar system constituted by pairs of elements like x and X_1, does admit of a real auxiliary conic, which, for projective reasons, is an ellipse. From the foregoing, it is clear how, having determined the ellipse of elasticity of an elastic system, the center of instantaneous rotation of the terminal point of the system for any force rigidly applied to its terminal point, is readily found.

15.—Elastic Weight.—Let the force, R_1 (Fig. 12), be decomposed into two others, one equal and parallel to R_1, passing through G, the center of the ellipse described, the other infinitely small, with its line of action at infinity (corresponding to a couple). The first component will cause a rotation about the corresponding anti-pole of the diameter on which the component lies. As previously mentioned, however, the anti-pole of a diameter lies at infinity on the conjugate diameter, therefore, this component will cause a displacement of the point on a straight line perpendicular to the conjugate diameter of its line of action. The second component, which is equivalent to a couple the moment of which is $M = R_1 . r$, will cause a rotation through an angle, ϕ, about the center of the ellipse, as this center is the anti-pole of a line at infinity, ϕ being proportional to M, whence: $\phi = G . M$. The constant, G, is called the "elastic weight" of a given elastic system. If this elastic weight, G, is considered as spread all over the construction, and its center of gravity as the center of the ellipse, the following theorems hold good.

16.—Considering the elastic weight, G, as applied at the center of the ellipse and substituting for M its value, $\phi = R \cdot r \cdot G$. Expressed formally this means that:

A terminal point, A, of a plane elastic system acted on by a force, R, rigidly connected with it, will rotate about the anti-pole (with respect to the ellipse of elasticity of the system) of the line of action of R, the angle of rotation being equal to the product of R and the statical moment of the elastic weight, G, with respect to the line of action of R.

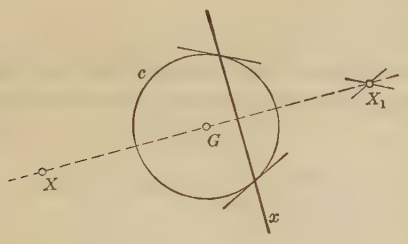


FIG. 13.

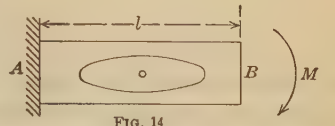


FIG. 14

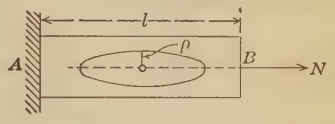


FIG. 15.

17.—The displacement, δ, of the point, A, in a direction, $A\,v$, is equal to the product of the angle of rotation, ϕ, and the distance, d, between the center of rotation and the direction line, $A\,v$. (See Paragraph 12). Whence $\delta = R \cdot r \cdot d \cdot G$. That is:

The displacement of a point, A, in a direction, $A\,v$, due to a force, R, is equal to the force multiplied by the product of inertia of the elastic weight taken with respect to the lines, R and $A\,v$.

18.—If the line, $A\,v$, coincides with the line of action of the force, the product of inertia becomes the moment of inertia taken with respect to the line, $A\,v$.

19.—Calculation of Elastic Weights.—Let $A\,B$, Fig. 14, be a fractional part or element of an elastic system acted on by the moment, M. (These elements are to be taken small enough to be considered of constant cross-section, and are to be regarded as fixed at one end and free at the other.) The virtual work done by this moment will be: $L = M \cdot \phi$, in which, ϕ is the angle of rotation of the terminal section, B, with respect to the section, A. From the theory of deflection:

$$\phi = \frac{M \cdot l}{E \cdot I}$$

Therefore, substituting the value of ϕ given in Paragraph 15,

$$L = M \cdot \phi = M^2 \cdot G = \frac{M^2 \cdot l}{E \cdot I}$$

Hence,

$$G = \frac{l}{E \cdot I}$$

which formula suffices for calculating the elastic weights.

20.—Calculation of Minor Axis of the Ellipse of Elasticity.—Let the element, *A B*, Fig. 15, be stretched by a force, *N*, acting in the geometrical axis of *A B* at the face, *B*. Its virtual work will be:

$$L = N \,.\, \Delta\, l = N \,.\, \frac{N \,.\, l}{E \,.\, A} = \frac{N^2 \,.\, l}{E \,.\, I} \,.\, \frac{I}{A}$$

in which A = area of cross-section. Substituting the value of G given in Paragraph 19:

$$L = N^2 \,.\, G \,.\, \frac{I}{A}$$

ρ being the semi-axis of the ellipse of elasticity, as shown in Fig. 15, the displacement of the face, *B*, in view of the principles given in Paragraph 17, is, $N \,.\, G \,.\, \rho^2$; therefore:

$$L = N \,.\, N \,.\, G \,.\, \rho^2$$

As, however, the virtual work is the same in both cases:

$$N^2 \,.\, G \,.\, \frac{I}{A} = N^2 \,.\, G \,.\, \rho^2$$

Therefore:

$$\rho = \sqrt{\frac{I}{A}}$$

which is the semi-axis of the ellipse of elasticity normal to the geometrical axis of the element, *A B*.

21.—Calculation of the Major Axis of the Ellipse of Elasticity.—Let *A B* in Fig. 16, be the same element of the system considered previously, and *T*, perpendicular to the axis, a force passing through the center of the ellipse, and rigidly connected with the face, *B*. The virtual work becomes:

$$L = T \,.\, \delta$$

which, in the light of Paragraph 17, becomes:

$$L = T \,.\, T \,.\, G \,.\, {\rho_1}^2$$

in which, ρ_1 is the semi-axis shown in Fig. 16.

Dividing the solid, *A B*, into elements, $\Delta\, x$,

$$M = T \,.\, x$$

in which, x is the distance between any one of these elements and the center of the ellipse. By the usual formula for virtual work:

$$L = \int \frac{M^2 \,.\, d\, x}{E \,.\, I} + \int \chi \, \frac{T^2 \,.\, d\, x}{S \,.\, A}$$

Integrating between $-\frac{1}{2}\, l$ and $+\frac{1}{2}\, l$, this will become:

$$L = \frac{T^2 \,.\, l^3}{12 \,.\, E \,.\, I} + \chi \, \frac{T^2 \,.\, l}{S \,.\, A} = T \, \frac{T \,.\, l}{E \,.\, I} \left(\frac{l^2}{12} + \frac{E}{S} \cdot \chi \cdot \rho^2 \right)$$

in which, S is the modulus of elasticity in shear and χ is a numerical constant, depending only on the cross-sectional shape.

From the equation:

$$\rho_1 = \sqrt{\frac{L = T \,.\, T \,.\, G \,.\, \rho_1^2}{\frac{l^2}{12} + \frac{E}{S}\,\chi\,\rho^2}}$$

Assuming that $S = \infty$, as is always permissible in arch design, that is, neglecting shear, the preceding formula becomes:

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

which is the value of the semi-axis of the ellipse of elasticity along the geometrical axis of the solid.

21 (*a*).—In the case of lattice work, of the type shown in Fig. 16 (*a*), by substituting in the preceding formulas the area, $2\,A$, of the cross-sections of the upper and lower flanges of the element, and the expression, $\frac{1}{2}$, $A \,.\, h^2$, for I:

$$G = \frac{2\,l}{E \,.\, A \,.\, h^2};\ \rho = \frac{h}{2}*$$

Furthermore, the displacement caused by a vertical force, 1, is given, according to Paragraph 18, by:

$$1 \,.\, G \,.\, \rho_1^2$$

On the other hand, the displacement must consist of two terms: The first due to the flanges and derived as the similar term already found for a solid element when $I = \frac{1}{2}\,A \,.\, h^2$, which term is $\frac{l^3}{6\,E \,.\, A \,.\, h^2}$. The other term is due to the lattice work: the strain is $1 \,.\, \frac{s}{h}$; therefore, the deformation is $\frac{1\frac{s}{h} \,.\, s}{E \,.\, A'}$ for each element of the lattice, in which, A' is the area of the cross-section of the element of the lattice and the corresponding vertical displacement is given by $1 \,.\, \frac{s^3}{E \,.\, A' \,.\, h^2}$. Noting that the total number of members of the lattice work is $\frac{l}{f}$,

$$G \,.\, \rho_1^2 = \frac{l^3}{6\,E \,.\, A \,.\, h^2} + \frac{s^3 \,.\, l}{E \,.\, A' \,.\, f \,.\, h^2} = \frac{2\,l}{E \,.\, A \,.\, h^2}\left(\frac{l^2}{12} + \frac{A \,.\, s^3}{2\,A' \,.\, f}\right)$$

Therefore,

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \,.\, s^3}{2\,A' \,.\, f}}$$

Finally, in the case of a lattice work of the type shown in Fig. 16 (*b*), to the expression of ρ_1 already given, must be added another term due to the uprights of the lattice work, and this term is derived from the one already

* See Paragraphs 19 and 20.

found for the diagonals by putting A'' in the place of A' and h in the place of s, so that:

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{A \cdot s^3}{2\,A' \cdot f} + \frac{A \cdot h^3}{2\,A'' \cdot f}}$$

As before, disregarding the deformation due to the web members of the lattice, this value becomes:

$$\rho_1 = l\sqrt{\frac{1}{12}}$$

21 (b).—For other types of lattice work, in which the flanges are not parallel, it is generally convenient to consider separately the elastic weights of the single members, and apply them to the corresponding poles (joints).

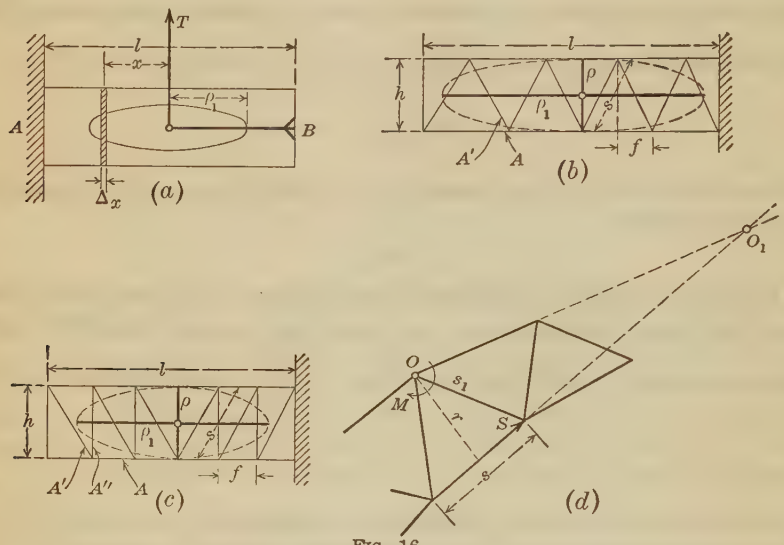


FIG. 16.

The elastic weight of any member, s, Fig. 16 (c), to be applied to its relative pole, O, is found by considering a rotation through an angle, ϕ, about the point, O, which is given by:

$$\phi = \frac{S \cdot s}{E \cdot A \cdot r}$$

in which, S is the strain in the member, s, caused by the moment, M. Noting that $M = S \cdot r$:

$$\Delta G = \frac{\phi}{M} = \frac{S \cdot s}{E \cdot A \cdot r} \div S \cdot r$$

that is,

$$\Delta G = \frac{s}{E \cdot A \cdot r^2}$$

In the same manner, the elastic weight of a member, s_1, of the web can be determined and this elastic weight should be applied to the pole, O_1. In most

engineering problems it is not necessary to take into consideration the deformations of the web members of a system. The ellipse of inertia of the elastic weights, ΔG, assumed to be applied to the poles of a lattice system, constitutes its ellipse of elasticity.

22.—Elementary Applications.—In the design of an arch, one end is assumed to be free and the other fixed, the ellipse of elasticity with respect to the free end being determined. The elastic weight of each voussoir (calculated as explained in Paragraph 19), considered as a force in accordance with the preceding principles, is applied at the center of gravity of the corresponding voussoir or element into which the arch is divided, and the axes of each elementary ellipse are determined in position and magnitude (see Paragraphs 20 and 21). Then, the ellipse of inertia of these elastic weights, constructed by the same methods used in graphic statics to find the ellipse of inertia, of areas, or of forces, etc., will be the ellipse of elasticity relative to the free end of the system. In other words, the ellipse of elasticity is simply the ellipse of inertia of the elastic weights.

23.—Let $A\ B$ in Fig. 17 be an elastic arch, with a fixed end at B and a free end at A, divided into an arbitrary number of voussoirs. Now, let a force, F, rigidly connected with the terminal point, A, act on the system. It will cause a rotation of the section, A, with respect to the section, B, the value of which will be:

$$\phi = F \,.\, r \,.\, \Sigma \frac{\Delta s}{E \,.\, I} = F \,.\, r \,.\, G^*$$

in which, $\Delta s = l =$ the length of the voussoir, and $\Sigma \dfrac{\Delta s}{E \,.\, I} =$ the sum of all the elastic weights, that is, the total elastic weight of the arch, G.

The rotation through the angle, ϕ, will be about D, the anti-pole of F with respect to the ellipse of elasticity of the whole arch. (See Paragraph 14.)

24.—The displacement, δ, of A along any given direction, $A\ x$, is, by Paragraph 17:

$$\delta = F \,.\, r \,.\, d \,.\, G$$

in which, $d =$ the distance from the direction line, $A\ x$, to the center of rotation, or anti-pole, D, and $r =$ the distance from the center of the ellipse to the line of action of F.

If the arch were fixed at A and free at B, and the force, F, were rigidly connected to Section B, then Section B would rotate, with respect to Section A, through the same angle as before, and about the same point, D.

25.—When the force, F, Fig. 18, passes through the center of gravity, G, of the system, the point, A, moves (Paragraph 6), on the normal, $A\ a$ to y, the conjugate diameter of the line of action of F with respect to the ellipse. It is, therefore, only when a force, F, acts along an axis of the ellipse that the displacement of A is parallel to the line of action of the force (as the axes are the only conjugate diameters of the ellipse perpendicular to each other). The expression for δ, given previously, becomes indeterminate as $d = \infty$ and $r = 0$.

* See Paragraph 16.

The component of the displacement of A in the direction, $A\,x$, however, may be given also by the expression:

$$\delta = F\,.\,r\,.\,d\,.\,G$$

in which, d = the distance from the direction line, $A\,x$, to the center of the ellipse, and r = the distance from the line of action of F to the center of rotation, or anti-pole, X, of the direction line, $A\,x$. (See Paragraph 5.)

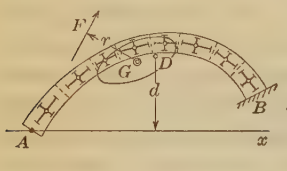


FIG. 17.

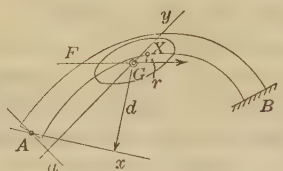


FIG. 18.

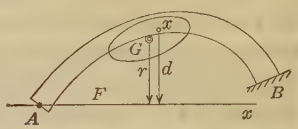


FIG. 19.

26.—If F acts along $A\,x$, Fig. 19, the displacement, δ, along this line itself is given by the product of the force, F, and the moment of inertia of the elastic system with respect to the line, $A\,x$; but from Paragraph 6, the expression of δ given in Paragraph 24 remains unchanged, therefore:

$$\delta = F\,.\,r\,.\,d\,.\,G$$

Application to a Trussed Arch Rib

27.—The preceding principles will now be applied to the case of a trussed arch rib, as this is simpler than the case of a solid rib, and as the extension of the general method to the solid arch depends on it. The treatment, however, will be incidental, and only those points of detail that are of interest in the solid arch theory will be carried out in complete working detail.

Fig. 20 represents an arch with fixed ends for which the ellipse of elasticity is to be drawn. Also, there are to be constructed five polygons (funicular polygons).

28.—*Polygon* p_1.—The elastic weights, w, are treated as vertical forces applied at their respective poles, 1, 2, 3, . . . 11. The eleven elastic weights, w_1, w_2, w_3, . . . w_{11}, are laid off on a vertical, and a pole, P_1, is chosen the distance of which from the vertical is equal to the length of the vertical, that is, $\Sigma\,w$. The funicular polygon, p_1, corresponding to this pole and load line is then laid off to connect the verticals through the poles, 1, 2, 3, . . . 11. The vertical, y (resultant), through the intersection of the first and last sides of the polygon, p_1, will, by ordinary graphics, contain the center of gravity of the system of elastic weights. Furthermore, the sides of the polygon, p_1, produced, will determine on this vertical, segments proportional to the statical moments of the corresponding elastic weights with respect to this same vertical through the center of the system.

A vertical drawn through any pole (the poles or centers of rotation of a truss are the joints)—for example, Pole 4, Polygon p_1—will cut the polygon in two points; the segment, η_0, between these points, is proportional to the statical moment of all the elastic weights to the right of Pole 4 with respect to this vertical.

29.—Polygon p_2.—By the methods of graphic statics determine the axis, x, conjugate with the axis, y (see Paragraph 5 for definition of conjugate axes). At the same time, determine G, the center of gravity of the system of elastic weights at the intersection of the x- and y-axes.* Again, treating the elastic weights as forces applied at the points, 1, 2, 3, . . . 11, this time, however, parallel to the conjugate axis, x, lay off the eleven elastic weights on a line parallel to x, assume a pole, P_2, on y for convenience and, connecting the lines through 1, 2, 3, . . . 11, parallel to x, draw the corresponding funicular polygon, p_2. The center of gravity of elastic weights acting in this direction lies on a line parallel to x through the intersection of the first and last sides of the polygon, p_2. The intersection of this line with the line, y, again gives the center, G, of the system.

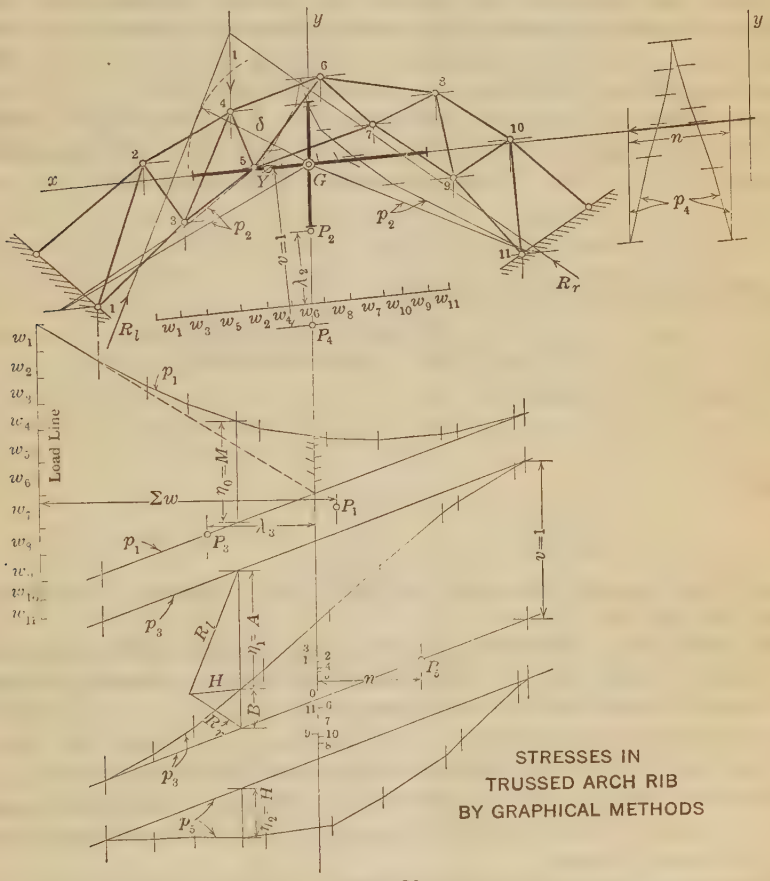


FIG. 20.

30.—Polygon p_3.—Taking a pole, P_3, at an arbitrary distance, λ_3, from y, and considering the statical moments with respect to this vertical, y, as vertical

* In Fig. 20 this construction has been made, assuming any vertical axis, y', and finding its relative pole, Y.

forces applied to the same respective poles, 1, 2, 3, . . . 11, construct the funicular polygon, p_3. Then, the expression, $\Sigma w \cdot \lambda_3 \cdot v$, will be the moment of inertia of the elastic weights with respect to the axis, y, where v is the distance measured parallel to y between the first and last sides of the polygon, p_3. (The values, Σw, λ_3, and v correspond, respectively, with b, c, and Σn of Paragraph 3.)

31.—Polygon p_4.—Just as the sides of p_1 intercept on y-segments proportional to the static moments with respect to y of the elastic weights, so the sides of p_2 intercept on x-segments proportional to the static moments with respect to the axis, x, of the elastic weights, when they act in the direction parallel to x. Treating such segments as forces parallel to x applied at the poles, 1, 2, 3, . . . 11, previously used, assume a pole, P_4, at a distance, v, from x, and construct the corresponding funicular polygon, p_4; this will make the moment of inertia of the elastic system with respect to the axis, x, equal to $\lambda_2 \cdot v \cdot n$ (see Paragraph 3, as before), in which n is the distance between the first and last sides of p_4 measured parallel to x.

32.—Ellipse of Elasticity.—The quantities thus far obtained, admit of the drawing of tangents to the ellipse of elasticity of the system, parallel to the axes, x and y. Their normal distances from x and y are:

$$\sqrt{\frac{\lambda_2 \cdot v \cdot n}{\Sigma w}}, \text{ and } \sqrt{\frac{\Sigma w \cdot \lambda_3 \cdot v}{\Sigma w}} = \sqrt{\lambda_3 \cdot v}$$

corresponding to the general formula:

$$\rho = \sqrt{\frac{I}{A}} = \sqrt{\frac{\text{Moment of inertia of forces}}{\text{Sum of forces}}}$$

and, if λ_2 is taken so that,

$$\frac{\lambda_2}{k} = \Sigma w$$

then,

$$\sqrt{\frac{\lambda_2 \cdot v \cdot n}{\Sigma w}} = \sqrt{\frac{k \cdot \lambda_2 \cdot v \cdot n}{\lambda_2}} = \sqrt{k \cdot v \cdot n}.$$

From these conjugate diameters, the ellipse may be drawn. The results are shown on Fig. 20.

33.—Polygon p_5.—Polygon p_2 gives the static moments of the elastic weights with respect to the axis, x. Apply these eleven static moments, considered as forces acting vertically, to the same poles, 1, 2, 3, . . . 11; lay them off on a vertical, and assume a pole, P_5, distant n (Polygon p_4), from this vertical, and connect their lines of action by the funicular polygon, p_5. Its first and last sides coincide, because the directions of x and y are conjugate (see Paragraph 5), and, therefore, the product of inertia with respect to the two axes must be zero.

Polygon p_5 will intercept on a vertical through any pole, for example, Pole 4, a segment, η_2, proportional to the product of inertia of elastic weights to the right of Pole 4. Its value is $\lambda_2 \cdot n \cdot \eta_2$. (See Paragraph 2.)

34.—Assuming now a load, 1, to be applied at any point, for example, Pole 4, the left end of the arch being free, and the right end fixed, the force, 1, will dis-

place the free end of the arch and any point rigidly connected therewith, for example, G. Such displacement may be conceived as composed of a rotation, a vertical displacement along y, and a displacement along x.

35.—The rotation of G is equal to the force, 1, multiplied by the sum of the static moments of all the elastic weights to the right of the line of action of the force with respect to this line of action. (See Paragraph 16.) If η_0 (Polygon p_1) is the segment proportional to the static moment on this vertical, the rotation will be $1 \cdot \Sigma w \cdot \eta_0$.

36.—The vertical displacement of G is equal to the product of the force, 1, by the moment of inertia of all the elastic weights to the right of Pole 4, with respect to the vertical through Pole 4. If η_1 (Polygon p_3) is the segment proportional to this moment of inertia, then, by Paragraphs 17 and 30, its value is $1 \cdot \Sigma w \cdot \lambda_3 \cdot \eta_1$.

37.—The displacement of G along x is given by the product of the force, 1, and the product of inertia of the elastic weights at the right of Pole 4 with respect to the line of action of the force, 1, and to the axis, x (Paragraphs 17 and 33). Hence, if η_2 (Polygon p_5) is the segment proportional to the product of inertia, the actual value of this product will be $\lambda_2 \cdot n \cdot \eta_2$, and the displacement will be $1 \cdot \lambda_2 \cdot n \cdot \eta_2$.

38.—*Reactions.*—To force the free end (left end of the truss) to return to its original position, will require the application of a reaction to the free end. The reaction, of course, will force back the point, G, rigidly connected with the free end of the arch, and may be considered as the resultant of a moment, M, about G, a vertical reaction, A, along y, and a reaction, H, along x.

39.—The rotation of Point G due to the moment, M, and in terms of the elastic weight, G, is given by G, M, which is equivalent to $\Sigma w \cdot M$; but the rotations of G, one due to the force, 1, and the other due to the reaction moment, M, must be the same, therefore:

$$\Sigma w \cdot M = 1 \cdot \Sigma w \cdot \eta_0$$

or,

$$M = 1 \cdot \eta_0$$

40.—The vertical displacement of G due to the vertical reaction, A, is equal to the product of the force, A, and the moment of inertia of the whole elastic system with respect to the vertical through G, namely, $\Sigma w \cdot \lambda_3 \cdot v$. Then, $A \cdot \Sigma w \cdot \lambda_3 \cdot v$ will be such displacement, and it must be equal to that previously found, or,

$$A \cdot \Sigma w \cdot \lambda_3 \cdot v = 1 \cdot \Sigma w \cdot \lambda_3 \cdot \eta_1$$

Supposing that v is, to scale, the unit of force,

$$A = \eta_1$$

41.—The displacement of G along x is given by the product of the force, H, and the moment of inertia of the elastic weights of the whole system with respect to the axis, x, that is, by $H \cdot \lambda_2 \cdot v \cdot n$. Putting this equal to the quantity previously found:

$$H \cdot \lambda_2 \cdot v \cdot n = 1 \cdot \lambda_2 \cdot n \cdot \eta_2$$

Assuming, again, that v is the unit of force,

$$H = \eta_2$$

42.—Influence Lines.—It is evident, therefore, that the three polygons, p_1, p_3, and p_5, are the influence lines of the three quantities M, A, and H.

43.—Position of Reactions.—The first and last sides of Polygon p_3 are extended intercepting on the vertical through the load segments which are equal to the vertical reactions at the supports (Fig. 20). It is then easy to find the reactions, R_l and R_r, by combining the vertical components with the horizontal thrust already found, as shown on Fig. 20.

To find the actual position of the two reactions with respect to the arch, the moment, M, of the force acting is given with respect to the center, G, hence:

$$M = 1 \cdot \eta_0 = R_l \cdot \delta$$

in which, δ is the lever arm of the moment, therefore:

$$\delta = \eta_0 \cdot \frac{1}{R_l}$$

Hence, a tangent parallel to R_l obtained from Polygon p_3 to a circle of radius, δ, with center at G, will be the line of action of R_l. The other reaction, R_r, may be obtained similarly, or, alternatively, by proceeding from the fact that the line of action of R_r must pass through the intersection of the vertical of the load, 1, and the line of action of R_l, and be parallel to R_r in Polygon p_3. These two lines of action may also be obtained by other methods.

44.—Intersection and Envelope Lines.—The geometrical locus of the intersections of R_l and R_r is called the "line of intersection"; the curve tangent to the left reactions is the "left envelope line", and the similar curve on the right is the "right envelope line."

Application to a Solid Arch Rib

45.—As far as deformation is concerned, a latticed rib may be substituted for a solid rib, as will become clear from the following demonstration.

Consider the solid rib to be divided into elementary lengths, ds, with N and M, respectively, a normal force and its moment with respect to the middle cross-section of one of these elements. N is the resultant of the external forces applied to the arch to the left, for example, of the section considered. The shear, as is customary, is neglected as being very small for arches.

The deformation, $\epsilon \cdot ds$, due to the elasticity of the material and temperature changes, is given by the equation:

$$\varepsilon \cdot ds = \frac{N \cdot ds}{E \cdot A} + a \cdot t \cdot ds$$

and the angle, $d\theta$, of rotation of the terminal section of the element with respect to its first section is:

$$d\theta = \frac{M \cdot ds}{E \cdot I}$$

in which,

N = the normal force;
E = the modulus of elasticity;
A = the area of cross-section;
a = the coefficient of expansion;
t = the number of degrees of temperature change;
M = the moment; and
I = the moment of inertia of cross-section.

Imagine $u_1\ u_2$ and $v_1\ v_2$, Fig. 21, to be the geometrical axes of the upper and lower flanges of a certain hypothetical latticed arch, all material of the solid section being assumed to be concentrated in these flanges. Furthermore, $u_1\ u_2$ and $v_1\ v_2$ are assumed to be in the same plane with the force and parallel to the geometrical axis of the element considered. Let $\rho = \sqrt{\frac{I}{A}}$ be their distance from the axis; also, let v be the middle point of $v_1\ v_2$, which will be assumed to be the pole of $u_1\ u_2$, so that the lattice shall consist of $u_1\ v$, $v\ u_2$; similarly for the point, u.

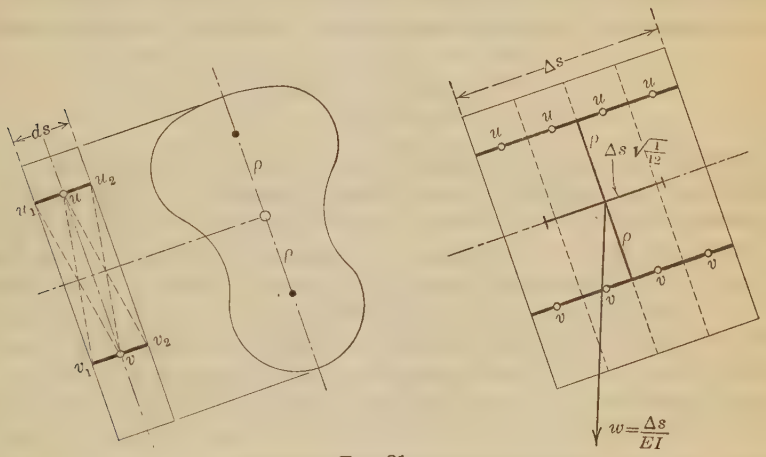


FIG. 21.

The stresses, therefore, in the flanges, $u_1\ u_2$, $v_1\ v_2$, due to the force, N, and the moment, M, with respect to the middle section of the element, $d\,s$, will be:

$$-\frac{N\rho + M}{2\rho} = -\frac{N}{2} - \frac{M}{2\rho}$$

$$-\frac{N\rho - M}{2\rho} = -\frac{N}{2} + \frac{M}{2\rho}$$

and the deformation of the panel, $u_1\ u_2$, $v_2\ v_1$, of the hypothetical latticed arch, considering only the relative displacement of the terminal sections of the element, $d\,s$, will be the same as for the solid element. In fact, due to the normal force, N, and the change of temperature, t^0, the points, u_2 and v_2, will be dis-

placed with respect to the points, u_1 and v_1, in a direction parallel to the geometrical axis of the element, $d\,s$, and such displacement will be equal to:

$$-\frac{\frac{N}{2}\,.\,d\,s}{E\,\frac{A}{2}}+\alpha\,.\,t^0\,.\,d\,s=-\frac{N\,.\,d\,s}{E\,.\,A}+\alpha\,.\,t^0\,.\,d\,s=\varepsilon\,.\,d\,s$$

and due to the moment, M, the line, $u_2\ v_2$, will rotate with respect to the line, $u_1\ v_1$, about its middle point through an angle given by:

$$\frac{1}{\rho}\,.\,\frac{\frac{M}{2\,\rho}\,.\,d\,s}{E\,.\,\frac{A}{2}}=\frac{M\,.\,d\,s}{E\,.\,I}=d\,\theta$$

so that the evaluation of the statically indeterminate quantities and the deformations of the elastic arch with solid cross-section is always possible by the substitution of the upper and lower flanges of a hypothetical latticed arch.

Moreover, it is legitimate to combine sufficient infinitesimal elements, $d\ s$, to obtain a sum, $\Delta\ s$, such that it may be regarded as a prismatic solid (Fig. 21), and then consider the centers of gravity of these elements, $\Delta\ s$ (centers of the points, u and v, for integral element, $\Delta\ s$), as loaded by the elastic weights:

$$\int 2\,.\,\frac{d\,s}{E\,\frac{A}{2}\,(2\,\rho)^2}=\frac{\Delta\,s}{E\,.\,I}=w$$

provided that, after constructing the ellipse of elasticity (or, rather, the axes, as these only are needed) for each element (voussoir), $\Delta\ s$, corresponding to the points, u and v, of that element, the statical moments of the w's, regarded as new forces, shall, in obtaining the moments of inertia and products of inertia of the w's, be applied, not to the centers of gravity of the elements, $\Delta\ s$, but to the anti-poles of that axis with respect to the ellipse of elasticity of that voussoir. This follows from the fact that the anti-poles are simply the centers of gravity of the elastic moments. (See Paragraphs 4, 5, and 6.)

46.—The stresses in the arch, due to changes in temperature, may be easily treated by this method, but the explanation will be omitted here, for the paper presented has not dealt with this part of the design.

DISCUSSION

J. Charles Rathbun,* M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to the literature on the analysis of the concrete arch ring, containing as it does an Appendix describing the method of analysis by means of the ellipse of elasticity.

One important feature of the paper is that it considers the elasticity of abutments and foundations. Actually, abutments are elastic, likewise foundations. Designers are too prone to compute the stresses of the ring with great accuracy under the assumption that the abutments are rigid. As the central span is designed first with rigid abutments and, later, with elastic abutments, some idea of the effect of this change can be gained. Incidentally, the most important part of the structure is the abutment. Nearly all arch failures can be traced to movement of the abutments, whereas in the drafting-room most of the time and study are given to the ring.

This method of arch analysis is especially adaptable to the investigation of the effect of moving and concentrated loads, but even in the investigation of stresses due to dead loads it is more rapid in application than the more popular method given in "Principles of Reinforced-Concrete Construction", by Turneaure and Maurer. On several occasions, the writer has analyzed arch rings by one of these methods and used the other as a check. A draftsman inexperienced in arch design requires about the same amount of time for the analysis by either method when working under an experienced engineer. However, a man familiar with both methods would save one-third to one-fifth of his time by using the author's method. A large percentage of the time used in Turneaure's method is taken in dividing the arch ring accurately into segments of varying length.

This shows that the author's method requires more study before it can be applied than that of Professor Turneaure, but that it is more rapid in its application. Of course, by means of legitimate approximations the time involved in applying the latter method may be considerably shortened. In view of the fact that an engineer needs to master the theory but once, after which he may apply it many times, it is surprising that the graphical method has not proven more popular.

The writer has studied this method with a view to discovering the reason for its lack of popularity. With the exception of poles and polars, the author uses only simple graphical constructions that should be familiar to every structural designer. The author has developed the ellipse of elasticity using the method of Projective Geometry and the Principal of Reciprocity, rather than that of Analytic Geometry. This is the more elegant method from a mathematical standpoint, but the curriculum of an American engineering school does not include this branch of mathematics. Although poles, polars, ellipses, conjugate axes, etc., are taught in the undergraduate engineering courses in Analytics, it must be admitted that they are totally forgotten before the student obtains his diploma. The subject could be quickly reviewed, if presented in

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the language of Analytic Geometry. There is danger that one reading a paper like that of the author assumes that it involves mathematics more advanced than he has acquired or is willing to acquire and, therefore, leaves it unread. Such expressions as "the auxiliary conic of the system is imaginary" are discouraging to any one, even the most enthusiastic theorist.

In explaining this theory, the writer has been in the habit of avoiding any reference to the ellipse as long as possible, bringing it in only after an analytic relationship between the line of force and the center of rotation has been established. To do this two examples are first solved, the first of which is a study of the location of the center of rotation of the end of a simple cantilever beam when deflected by a force applied at the end, but acting along a given line.

Let Fig. 22 represent a horizontal straight cantilever beam the length of which is l, deformed by a force, F, acting along the line, BC, but applied at the point, A. To find the co-ordinates, h and k, of the center of rotation, D, of the point, A, in terms of the equation of the line of force, BC, resolve the force, F, into its vertical and horizontal components, T_B and N, acting through B, the point where F intersects the horizontal axis of the beam (the X-axis).

The deflection of A due to T_B acting alone is:

$$\Delta = \frac{T_B\, l^2}{E\, I}\left[\frac{l}{12} + \frac{e}{2}\right] + \chi\,\frac{T_B\, l}{S\, A}$$

while the angle of rotation is $\phi = \dfrac{T_B\, l\, e}{E\, I}$, using the author's notation and the

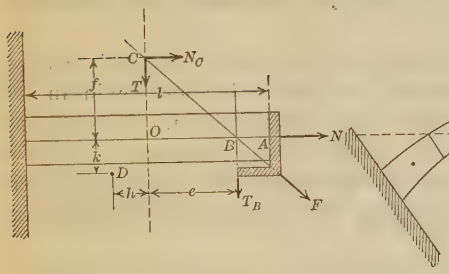

Fig. 22.

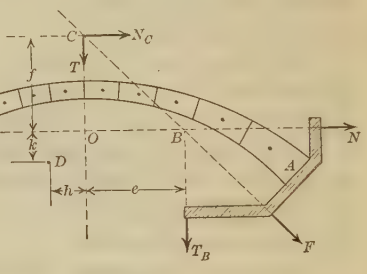

Fig. 23.

dimensions given in Fig. 22. Both these expressions have been derived in accordance with the common theory of flexure.

If the distance from the deflecting point, A, to the center of rotation is denoted by r (radius of rotation), it follows from Fig. 22, that $r = h + \dfrac{l}{2}$; but $\Delta = r\,\phi$, whence:

$$h + \frac{l}{2} = \frac{\Delta}{\phi} = \left\{\frac{T_B\, l^2}{E\, I}\left[\frac{l}{12} + \frac{e}{2}\right] + \chi\,\frac{T_B\, l}{S\, A}\right\} \div \frac{T_B\, l\, e}{E\, I}$$

Neglecting deformation due to shear, this reduces to $h\,e = \frac{l^2}{12}$. The deflection of the point, A, due to N acting alone, is $\frac{N\,l}{A\,E}$. There is no rotation.

When T_B and N act together, the point, A, rotates through an angle, $\phi = \frac{T_B\,l\,e}{E\,I}$, and at the same time moves vertically a distance, $\frac{l^2}{12\,e}$, and horizontally a distance, $\frac{N\,l}{A\,E}$. This amounts to a rotation through an angle, ϕ, about a center the co-ordinates of which are:

$$h = -\frac{l^2}{12\,e}, \text{ and } k = -\frac{N\,l}{A\,E} \div \frac{T_B\,l}{E\,I} = -\frac{N\,I}{T_B\,A\,e}$$

As h is entirely independent of the values of T_B and N, it follows that the center of rotation of the point, A, due to a force, F, acting through B, lies on the vertical line, $x = -h$. When N is zero, k is zero.

Again, resolving the force, F, into its vertical and horizontal components, T and N_c acting through C, the point where F intersects the vertical through the central point of the beam, the deflection due to T acting alone is obtained by placing $e = 0$ in the foregoing expressions obtained for T_B. $\Delta = \frac{T\,l^3}{12\,E\,I}$, and is vertical, while $\phi = 0$.

The horizontal deflection of the point, A, due to N_C acting alone, is $\Delta = \frac{N_C\,l}{E\,A}$, while $\phi = \frac{N_C\,f\,l}{E\,A}$; whence, $k = \frac{\Delta}{\phi} = \frac{I}{A\,f}$. As k is independent of the values of N_C and T, it follows that the center of rotation of the point, A, due to a force, F, acting through C, lies on the horizontal line, $y = -k$.

It follows that the co-ordinates, h and k of the center of rotation, D, of the point, A, when acted on by a force, F, along the line the equation of which is $\frac{x}{e} + \frac{y}{f} = 1$, are $-\frac{l^2}{12\,e}$ and $-\frac{I}{A\,f}$; and the graphical construction of the author's Paragraph 10 immediately follows.

Except for sign this is the relationship between the pole and polar of a real ellipse as given in standard texts on Analytic Geometry. The change of sign presents no difficulty.

Take as a more general example a curved cantilever beam (Fig. 23) made up of a series of parts the elastic weights of which are G. O is the center of gravity of the system of elastic weights and the X and Y-axes are drawn so that $\Sigma\,G\,x\,y = 0$. If the system is symmetrical these axes are best taken as vertical and horizontal.

The several movements of the point, A, due to the force, T_B, are:

$$\Delta\,y = \Sigma\,T_B\,G\,(x+e)\left(x+\frac{l}{2}\right) = T_B\,\Sigma\,G\,x^2 + T_B\,\frac{e\,l}{2}\,\Sigma\,G$$

$$\Delta\,x = \Sigma\,T_B\,G\,(x+e)\,(y+m) = T_B\,e\,m\,\Sigma\,G$$

$$\Delta\,\phi = \Sigma\,T_B\,G\,(x+e) = T_B\,e\,\Sigma\,G$$

whence,

$$h\,e = \frac{\Sigma\, G\, x^2}{\Sigma\, G} \cdot A\, s' \frac{\Delta\, x}{\Delta\, \phi} = m$$

the center of rotation for a vertical force lies on the axis. The other quantities and values may be obtained in a manner similar to those in the first example.

If the arch is unsymmetrical the axes may not be at right angles to each other. Taking advantage of the fact that the center of rotation due to a force parallel to one axis lies on the other, the second axis may be found after assuming the first.

Thus, the author's whole graphical method can be derived algebraically. Although this method is not as finished or as pleasing to those who have specialized in this type of theory as that given in the paper, it has the distinct advantage of presenting to the engineer a minimum of new ideas.

D. B. Steinman,* M. Am. Soc. C. E.—The method of the ellipse of elasticity provides a neat and most effective solution of problems in indeterminate structures. For the development of this theory and its various applications engineers are indebted to the late Professor W. Ritter, of Zurich, Switzerland, who devoted a lifetime to work in this field. Mr. Janni is deserving of credit for his clear and scholarly presentation of the general theory in this paper, and for having done more than any other engineer to introduce this method into American practice.

The speaker has been familiar with the method for many years, and has used it in the design of various indeterminate structures, including arches in steel and in reinforced concrete. Another practical problem in which this method yields a solution of striking advantage is the construction of deflection influence lines for the determination of maximum deflections of long-span structures. Applications of the elastic ellipse to the analytical and graphical analysis of arch ribs are also presented in the volume on "Plain and Reinforced Concrete Arches", which the speaker translated from the work by Professor J. Melan.

The author has clearly presented the standard method of applying the ellipse of elasticity to the graphic analysis of arch ribs. There is another method for solving the problem, also based on the elastic ellipse, but far more direct in its application, giving the influence line for any member or section of a hingeless arch rib directly by the construction of a single funicular polygon. It dispenses with the long string of preliminary funicular polygons, with the construction of reaction intersection locus and envelope curves, and with the entire laborious procedure of drawing reaction lines for each panel load, scaling the intercepts, calculating the resulting moments for each section for each panel load, and then plotting the final results in the form of influence lines.

The direct method is as follows: To obtain the moment influence line for any panel point of an arch rib, consider a unit force applied along the anti-polar of that panel point with respect to the central ellipse; the result-

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ing deflection curve of the arch is the required moment influence line. This deflection curve is simply obtained as the funicular polygon of the static moments of the elastic weights with reference to the anti-polar.

The construction is simple for any particular influence diagram desired. In the case of the problem of the single arch treated in the paper, where only four sections are investigated, only four funicular polygons would have to be drawn. These would be the final influence lines.

This method is only another application of the basic principle which the speaker first published a number of years ago: "Every influence line is a deflection diagram."*

There is a minor point of nomenclature which deserves attention. Mr. Janni refers to the critical points of a section as "kernel points". The speaker has always referred to them as "core points"; and other writers have labeled them "kern points." Although it makes little difference which name is used, it is certainly desirable in the early stage of development of a subject to avoid future confusion by adopting a single name for a given idea. These various terms are attempts to translate the German word, "kern", which is defined by the dictionaries as "core" or "kernel". The speaker prefers the form "core points" for several reasons. These points possess no particular significance in themselves, but derive their importance from the fact that they are points on the boundary of a significant area which is suitably termed the "core area". The "core area" of any section is the area through which the resultant must pass in order that there shall be no tensile stresses in the extreme fibers. In this concept attention is focused on the area; and such an area is better represented by the word, "core," than by the words, "kernel" or "kern", particularly as the first name is less foreign and more in accord with familiar engineering terminology.

The title and introductory paragraph of the paper indicate that its professed object was to investigate the permissibility of some simplifications in multiple-arch analysis. A careful reading of the paper does not reveal what simplifications the author had in mind, nor how he has shown their permissibility.

The results presented by Mr. Janni prove what is generally known, that it is not permissible to disregard the elasticity of the adjacent spans in investigating the central arch of a three-arch system; but he has not touched on the broader and more interesting problem of the extent to which it is permissible to disregard the remaining spans in a system of more than three arches.

If the object of the paper was, as indicated in the "Conclusion", to show that it is unnecessary to consider loading on the outer spans in investigating a three-arch system, the author has not completed his demonstration; he has evidently overlooked the possibility (which may upset his conclusions) of the two outer spans being loaded simultaneously. To provide for this condition, the respective side-span influence lines presented in Plate IV would have to be added together (the influence area for Section IV_1 being thereby doubled) before the areas could be compared with the main-span influence areas. Until

* *Engineering Record*, November 25, 1916.

this has been done and certain details have been clarified, the only conclusion indicated by the author cannot be accepted as valid.

In "Plain and Reinforced Concrete Arches" (previously referred to), there are two chapters on "Arches with Elastic Abutments" and "Arches Continuous over Several Spans." In these chapters, the various problems of elastic piers and multiple-arch systems are treated. Several methods of analysis are given, of varying degrees of precision and simplification, ranging from the exact theoretical analysis to the simplest short-cuts for approximate calculation. The simplification (which Mr. Janni attributes to Professor Guidi) of considering only three spans of a multiple-arch system, is presented. Other simplifications are analyzed and applied, such as disregarding the transmission of moment and rotation effects to adjacent spans, and assuming that the elastic yielding at the ends of a span consists only of horizontal displacement due to H (the arch thrust).

Investigations indicate that the transmission of the H-effect through a series of arches is the most important feature of continuous arch action. The transmission of the angular change due to H, of the rotation and horizontal displacement due to the end moments, M, and of the displacement due to the vertical reactions, V, may generally be disregarded; but it is not safe, in general, to disregard the transmission of the horizontal displacement due to H, from the loaded arch through the adjacent spans to the spans beyond.

The permissibility of the various simplifications mentioned will depend, of course, on the degree of rigidity of the intermediate piers. This factor also affects the general economic problem of continuous arch systems.

The inherent economy of arch construction is produced by the effect of the horizontal thrust, H, in counteracting the ordinary bending moments due to vertical loading. In a multiple-arch system, the arch effect (represented by the value of H) is reduced by virtue of the continuity. In the extreme case, assuming the intermediate piers to be non-resisting, the value of H for a single loaded span is reduced in the inverse ratio of the number of spans. The result is a material increase in the bending moments calculated for a single span, since the relieving effect of arch action has been so much diminished; each additional span taken into consideration will augment the calculated bending moments in the loaded span. This tendency is modified, of course, by the resistance introduced by the intermediate piers. Consequently, the rigidity of these piers not only governs the question of permissibility of simplifications, but also affects the economy of the continuous design.

From this it appears that there are conditions under which it will pay to choose massive piers rather than elastic piers, so as to isolate the separate spans and thereby avoid the loss of economy due to continuity. The question of the economic dividing line for the choice, or the determination of the relative degree of rigidity of piers and arches most desirable, is a problem deserving further investigation.

GEORGE PAASWELL,* M. AM. SOC. C. E. (by letter).—A few notes on the method of determining the functions for the ellipse of elasticity may be of

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interest. It is apparent that two points, fairly close together, determine the conjugate axis. As this axis forms a base line for most of the work following, it seems better to determine its direction analytically. Referring to Fig. 20, let the x-axis make an angle, ϕ, with the horizontal and let x and y denote the co-ordinates of the points, 1, 2, ... 11, with respect to a rectangular system through G as an origin. Then, $\tan \phi = \frac{\Sigma w x y}{\Sigma w x^2}$, due regard being paid to the sign of the co-ordinates.

The anti-poles require considerable graphical construction. Their positions may likewise be determined analytically. In Fig. 8, let P be the anti-pole with respect to the x-axis (again selecting a rectangular system through G), and let u_x, v_x, be the co-ordinates, P-P_y and P-P_x, respectively; x and y are the co-ordinates of the center, O, with respect to the center of gravity of the arch, G. Let P' be the anti-pole with respect to the y-axis, and u_y, v_y, the corresponding co-ordinates of the anti-pole with respect to O. Let Q be the angle that the major axis of the ellipse makes with the x-axis of the arch. Then:

$$u_x = \frac{a^2 \sin Q}{y}; \quad v_x = \frac{b^2 \cos Q}{y}$$

$$u_y = \frac{a^2 \cos Q}{x}; \quad v_y = \frac{b^2 \sin Q}{x}$$

a and b are the major and minor semi-axes of the ellipse. Replacing a and b by their approximate values in terms of the arch data (L = the length of a voussoir, ρ = the radius of gyration):

$$u_x = \frac{L^2 \sin Q}{12\, y}; \quad v_x = \frac{\rho^2 \cos Q}{y}$$

$$u_y = \frac{L^2 \cos Q}{12\, x}; \quad v_y = \frac{\rho^2 \sin Q}{x}$$

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—It is difficult to discuss a paper of this nature without appearing hypercritical; the writer feels confident that Mr. Janni will understand his attitude. Truth is not a thing that can long be suppressed because of fear that the one who takes the other side of an argument may be hurt by the statement of that truth. Progress is not attained by allowing ideas to congeal and reach a state of alleged established fact, especially the ideas that logic and test prove to be controvertible. For example, among the ideas now threatening to "freeze," are, that a reinforced concrete arch is an elastic structure and that an arch abutment can be made fixed ended.

This paper is of value or of no value according as these ideas are true or false. It is pertinent, therefore, to look into the fundamentals on which the text is based. A great deal of the literature of reinforced concrete is built on these same two ideas, and until they can be substantiated all this literature must be appraised accordingly.

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One of the indictments against standard practice in reinforced concrete design previously noted by the writer,* was that reinforced concrete arches cannot be considered as elastic arches and should not be designed as such. Mention was also made of the error of merely assuming that the ends of an arch are fixed and then making no provision in the design for attaining this condition.

The mathematics of the elastic arch is very complicated and tedious and the tendency, where elaborate cloaks of figures are used, is to obscure any shortcomings in the assumptions, by reason of the neatness with which the mathematical problem is solved.

The economy of the elastic arch, considering the arch ring alone, makes its use attractive, particularly because of the assumption of fixed-endedness of the arch ring. Also, the idea is fostered in the designer's mind that because of the supposed accuracy of the method, high unit stresses—a small margin of safety between calculated stresses and ultimate—can be utilized.

The actual elastic properties of concrete, where straight beams are considered, are of little moment; for in spite of the elaborate methods of locating the neutral axis, tests vitiate all these efforts, and it is found experimentally to be close to the middle of the depth of the beam, regardless of the amount of steel reinforcement. The principles of good design would be in no way disturbed if an agreed fixed position of the neutral axis of a reinforced concrete beam were used for all percentages of reinforcement.

The modulus of elasticity of concrete varies considerably, even for specimens made of apparently the same kind of materials. It also varies for different intensities of stress, which means a variation in the same cross-section of a member in bending. The first of these inconsistencies could not be allowed for mathematically; the other, if the law of its change should be discovered, might yield to analysis by adding manyfold to the complexity of an already overburdened mathematical solution.

If an arch could be made of glass, annealed for several months, and then if it could be fully anchored to perfectly rigid abutments, it would fit the elastic theory; but since concrete falls short of true elasticity, even in small specimens, when it is poured in large masses it is subject to initial internal stresses that are totally unknown, although their existence is a fact that cannot be denied. Shrinkage in concrete and the non-homogeneity of the several batches are factors that make a concrete arch far from the truly elastic body that fits in unstressed condition the shape of its support, according to the fundamental assumptions.

Furthermore, steel reinforcement in a concrete arch contributes materially to the lack of true elasticity and the fact that it is unstressed in its original condition. Shrinkage in the concrete puts initial stress in the steel, and any calculations of equivalent moment of inertia based on unstressed steel and concrete are vitiated by this undisputed fact.

The whole fabric of the method of designing reinforced concrete arches as elastic structures stands or falls according as the deflection of a reinforced

* "Some Mooted Questions in Reinforced Concrete Design", *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 54.

concrete beam under test can or cannot be accurately determined by computation. Tests prove that this deflection cannot even be approximated by computation. Here, in a nutshell, is the proof that the elastic treatment of the reinforced concrete arch is improper.

Tests proving the impossibility of computing or predicting the deflection of a reinforced concrete beam have been cited by Professor G. Lanza,* in which two identical beams showed results almost 100% apart. This is not an unusual finding.

Most arch abutments have their foundations in earth. Earth foundations always settle more or less. The settlement of the abutment of an arch would not be uniform, as are harmless settlements in building foundations; it would be a rocking or rotation, by reason of the fact that the intensity of pressure is greater under one edge of the abutment than under the other. There is no way to avoid this condition, except by adding wasteful mass to the abutments. An exceedingly small rotation of the abutment of an arch vitiates entirely all assumptions concerning stresses and renders quite useless the elaborate fabric of figures utilized in the calculations.

The fact is that the mass of concrete that it is necessary to add to the abutments of a concrete arch is so great that it greatly exceeds the alleged saving in the ring of the arch due to considering that arch ring elastic and fixed-ended instead of inelastic and hinged-ended. This phase of the design of a reinforced concrete arch, namely, the proportions of the abutments for rigidity of position, is never considered, however, in the literature of the elastic arch.

What about the arch that is founded on a rock? At Bendigo, Australia, some years ago, a skew arch failed.† This arch had especially massive abutments and was securely founded on solid rock. J. Charles Rathbun, M. Am. Soc. C. E., has shown‡ that the thrust of a skew arch, theoretically, is of uniform intensity to the extreme point of the acute corner, if the arch be fixed-ended. This arch failed by crushing in the obtuse corner, proving that, as common sense would dictate, the thrust took the short course and created excessive pressure at the obtuse corner. If the theory is correct, this is a demonstration that even founding an arch on solid rock does not make a fixed-ended arch of it.

Clyde T. Morris, M. Am. Soc. C. E., states§ that tests on a skew arch in a laboratory show that the greater part of the reaction is close to the obtuse corner. In some cases the reaction at the acute corner is negative. Professor Morris points out that Hool‖ states that "skew arches may be treated exactly as right arches, the span being taken parallel to the center line of the roadway and not at right angles to the springing lines of the arch." Theoretical conclusions such as this, based solely on hypothetical elastic properties of concrete have been proven by laboratory tests and by experience with structures to be incorrect and dangerous.

* *Journal*, Am. Soc. Mech. Engrs., Mid-October, 1909.
† *Engineering Record*, April 16, 1910.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.
§ *Engineering News-Record*, April 20, 1922, p. 638.
‖ "Reinforced Concrete Construction", Vol. III, p. 43.

As a recent light on the unsoundness of the assumption of fixed-endedness in arches, either for square arches or skew arches, it has been pointed out* that in a discussion before the Society tests on a full-sized arch bridge (119-ft. span) proved that the rotation of the piers is perceptible, although the piers are founded on rock. The significance of this fact is tremendous; it means that a large amount of very intricate theory taking up large space in standard works and the publications of Engineering Societies is of no value. The writer has been publicly making this claim for seventeen years. The elastic theory of reinforced concrete arches is based on a premise that is false in its conception, which involves absolute fixed-endedness in the abutments. All designs made on the elastic theory are wrong in just the proportion that the dimensions and sizes are influenced by the dictates of this theory.

Forms settle, abutments settle and rotate, and concrete shrinks in setting. No designer has a right to hide behind the theory of perfect elasticity, when common sense would tell him to design his arch as a row of blocks, hinged-ended, and then anchor the arch into the abutment for additional safety, and to avoid skew arches as such no matter how pretty the theory looks.

Two notable cases of failures of arches,† both of which occurred in Germany, illustrate what may happen when ultra-theoretical ideas are carried to their logical conclusion. These arches were most carefully made to fit the theory; the designer provided cast hinges at the ends and at the crown. The arches promptly collapsed with little or no load on the ribs except their own weight. One failure is "attributed by a technical expert employed to investigate it, to the use of an improper lubricant on the faces of the hinge joints." It is ridiculous to believe that proper lubrication would have saved this structure. If the designer had used more common sense and less alleged theory—had anchored the ends into the abutment and continued the rods through the crown and poured concrete where these expensive cast shoes were used—the structures would doubtless be standing to-day.

E. H. Harder,‡ Assoc. M. Am. Soc. C. E. (by letter).—As a result of several years of acquaintanceship, the writer has been led to believe that the author considers the ellipse of elasticity to be a panacea for all the ills of the structural designer. In this belief, Mr. Janni seems to stand alone, because little has been written on that subject in this country.§ Two writers, Messrs. Moreell and Whited,§ have treated the ellipse of elasticity in purely an explanatory manner and have not proclaimed its supposed virtues. The ellipse of elasticity is also conspicuous by its absence from European literature, with few exceptions, as noted by the author.‖ Quite recently, in discussing the sub-

* *Engineering News-Record*, January 24, 1924, p. 159.

† One of these failures is described in *Engineering News-Record*, January 4, 1923, p. 35; the other is described in *Engineering News*, October 27, 1904, p. 374.

‡ Structural Engr., Concrete Steel Eng. Co., New York, N. Y.

§ "Designing of an Arch System", by A. C. Janni, M. Am. Soc. C. E., *Journal*, Western Soc. of Engrs., Vol. XVIII (May, 1913), and also his contribution to Vol. III of "Reinforced Concrete Construction", by G. A. Hool; "The Method of the Ellipse of Elasticity and Its Application to Continuous Arches on Elastic Piers", by S. Moreell, Jr., *Journal*, Western Soc. of Engrs., Vol. XXII (1917), p. 406; and "The Mathematical Theory of the Elastic Arch", by Willis Whited, M. Am. Soc. C. E., *Proceedings*, Engrs. Soc. of Western Pennsylvania, Vol. 32 (1916–17), p. 271.

‖ See p. 1160.

ject with a Swiss graduate from the Polytechnic in Zurich, where Professor W. Ritter held a chair for many years, the writer learned that the students of this renowned school learned little or nothing about the ellipse of elasticity. In Europe, it seems to be regarded as an analytical curiosity, and contrary to the author's statement* it is not extensively taught or used and it has no special virtues, which aid in the solution "of the most difficult problems of engineering where the analytical methods are found impractical."

Culmann is conceded to have been the founder of graphical statics. He was the first to show the relation between a force and string polygon and to recognize their proper use in problems of statics. He, however, in his enthusiasm, attempted to build up a system of graphical statics free from the influence of mathematics and sought to base his work on purely geometrical conceptions. As Euclidean geometry was not sufficient for his needs, he resorted to projective geometry which has never found favor with the majority of engineers. Consequently, when he revised his first book, he used the more acceptable analytical geometry. Ritter, who had been a pupil of Culmann, was equally desirous of keeping graphical statics on a purely geometrical basis and gave as his reason† that he did not believe it proper to label his work graphical statics unless it was kept on such a basis. Furthermore, Culmann applied his graphical statics to Poinsot's ellipse of inertia and used it extensively in problems involving moments of inertia and products of inertia, principal and conjugate axes, and the determination of core areas and stress distributions of sections under direct stress and bending. Ritter further expanded the same conceptions to problems involving deflections of entire structures because he believed that the "ellipse of elasticity", as he called it, could be used as a fundamental conception to offset the then increasing popularity of the laws of virtual displacements.

Since that time (1888) the general use of graphical statics seems to have fallen more and more into disfavor, and engineers who must design structures for a livelihood rather than for pleasure prefer a happy combination of analytics and graphics. It is very likely for this reason that the Engineering Profession has never fully accepted the ellipse of elasticity and has shelved it with other curios. However, the pendulum may swing in the other direction, the ellipse of elasticity may be dusted off, and again the polars will rotate about the anti-poles.

The chief application of the ellipse of elasticity is in the analysis of indeterminate arches with solid webs or trussed ribs. Even in this case its advantages are questionable. For trussed ribs, this method disregards the effect of the web members on the deflections of the structure. This is erroneous, as is clearly shown by George F. Swain, Past-President, Am. Soc. C. E., in discussing his paper entitled "On a New Principle in the Theory of Structures."‡

The author's application, however, is erroneous because he neglects all the web members. In solid beams, the deformation due to shear is often or

* See p. 1160.

† "Anwendungen der Graphischen Statik", Preface to Vol. I.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 705.

generally neglected because (1) it cannot be accurately determined, and (2) it is small. In trusses, the deformation due to shear may be determined as accurately as for the chords and it may be considerable. Professor Ritter cites a case in which the deflection due to the web members was 41% of the total. Mr. Janni, therefore, ought to classify the various structures in which the influence of the web members on the deflection of the structure is negligible and should not content himself with the single statement that: "In most engineering problems it is not necessary to take into consideration the deformations of the web members of a system."

In the case of the solid reinforced concrete arch or the solid web arch of the plate girder type, the only advantage of the method of the ellipse of elasticity over prevailing methods of analysis is that it automatically takes care of the stresses due to the shortening of the arch axis under dead and live loads. These so-called arch-shortening stresses are usually computed separately, but may also be automatically included in the usual methods of arch computation by adding in the denominator of the formula for the horizontal thrust a factor which adequately takes care of the added deflection due to arch shortening.*

This fact concerning the ellipse of elasticity, namely, that it takes care of the dual deflection of the arch rib due to bending moments and direct stresses, seems not to be clearly understood by the author. On page 1158, under "Temperature", he states:

"The effect of temperature change, as well as that of shrinkage in concrete due both to the setting of the concrete and to the dead load and live loads, is not presented in this paper. The problem has been solved only for the live load, and in a manner which, in the present state of knowledge, represents the most complete solution that can be desired."

He has committed the same error elsewhere.† If he will examine the papers by Messrs. Moreell and Whited, he will find it clearly stated that no further arch-shortening stress computations are necessary by the use of the ellipse of elasticity. In his "Anwendungen der Graphischen Statik", referred to previously, Professor Ritter is also clear on this point. Except for the double rotations due to bending moments and direct stress, there would be no antipoles to cause trouble and no minor axes of the elemental ellipses of elasticity.

Regarding the graphics of the method as illustrated on Plate III, there is nothing new or novel. The same set of polygons are also obtained graphically by other methods in vogue. It is advisable to avoid the method utilized by Mr. Janni because of the inaccuracies and awkwardness of some of the construction (for example, the small force polygon with a pole at p_5). Any error in this small force polygon may mean a large error in the influence line for the horizontal thrust, p_5.

* "Plain and Reinforced Concrete Arches", by Melan; translated by D. B. Steinman, M. Am. Soc. C. E.

† In his paper, "Designing of an Arch System", *Proceedings*, Western Soc. of Engrs., Vol. XVIII (May, 1913), and "Reinforced Concrete Construction", by G. A. Hool, Vol. III, p. 140.

In the application of the ellipse of elasticity to the problem of the multiple-arch system, as developed by Professor Camillo Guidi and Professor H. Lossier, and as explained in English by Professor Moreell and in the third volume of Hool's "Reinforced Concrete Construction", there is at first glance a decided advantage. Nearly all other methods of solving this problem necessitate the use of a number of simultaneous equations, consequently, any method which obviates such equations would be desirable; but here the writer wishes to call attention to the incompleteness of the problem. The author has selected the easiest case for his demonstration—a symmetrical arch supported on two equally elastic supports (end arch and pier). A complete solution would call for all the influence lines for the end arches and the attendant influence lines in the remaining two arches. If the outer ends of the arch system, as used by Mr. Janni, are to be fixed, the stresses in these arches will be different from those of the central arch, especially at the outer skewbacks.

In the "Conclusion" (page 1158), the author states that Professor Guidi has suggested that, of a long series of arches in one system, only three adjacent arches need be considered as the elastic system necessary for the analysis and that all arches on either side of this system can be neglected. The suggestion is a good one, but as every series of arches has two ends, the author has not completed his problem because he has not shown what occurs at the ends and in the end arches. To do so by the method of the ellipse of elasticity, would entail a greater amount of work than was necessary by the limited analysis given in the paper. Likewise, Mr. Janni's influence line for the base of the pier at Section *S-S* (Plate IV) holds only for loads on the center span, A_2. What shape has the influence line for a load on Span A_1 or Span A_3? From an analysis of a similar series of arches, but with a higher rise ratio, the writer finds that the stress in the outer skewback section may be 20% higher than the stress in the skewback sections of the center arch of the system and, therefore, he cannot agree with the author that no matter how many arches constitute a system, if all are designed according to the influence lines as drawn for the center arch, A_2, the system is safe. This also throws a doubt upon the correctness of the paragraph on "Dead Loads" (page 1158), although for practical purposes, the dead load stresses in the three arches may be considered to be equal, varying as far as the writer has investigated only a small percentage.

The author asks for simplifications; unfortunately, there are no simplifications for this method—once started, the designer must follow through the entire process. There are, however, other methods, which offer certain advantages of control. Those interested would do well to read "Der durchgehende gelenklose Bogen auf elastischen Stuetzen", by Ernst Pichl, which is an analytical method fully explained by an example.

As no designer would plan a system of arches for bridge purposes of the slender outlines suggested by Mr. Janni for the purpose of demonstration, it is regrettable that he did not show the effect on the influence lines of doubling or trebling the pier thicknesses. Thickening the piers would reduce the negative bending moments at the skewbacks and decrease the positive bending moments at the crown.

HARDY CROSS,* M. AM. SOC. C. E. (by letter).—This paper presents in somewhat extended form, the so-called "theory of the ellipse of elasticity" and applies it to the analysis of a continuous arch system under the attractive title of "Design and Permissible Simplifications".

Paper Treats Design in a Restricted Sense.—As the writer commonly understands the term, "design" is the choice of form and arrangement of parts of a structure, under the limitations that practical considerations impose on the values and disposition of loads and other stress-producing conditions. The mathematical determination of theoretical stresses he commonly calls "analysis". Thus interpreted, a study of the design of multiple-arch systems would at least involve (1) the economic rise ratio in the arch; (2) the economic proportions of the piers; (3) the proper span length; (4) the effect on the choice of working stresses of liability to foundation settlement, probability of simultaneous maximum stresses from foundation effects, temperature, shrinkage, and live load, and especially of the general reliability of this type of structure; and (5) the acceptance or rejection of "split" loading in which the live load is not continuous.

The Author's Thesis.—Evidently, the author interprets "design" more in the sense in which the writer uses "analysis", for he states that the aim of the paper is to show whether, for practical purposes, it is possible to simplify the "strictly theoretical method" of design. Further, he merely states that the arch was first considered as having fixed ends, "the design being made accordingly", and the analysis is then extended to include the effect of flexible supports. About one-half the paper proper deals with the fixed arch. The author further states that the paper is restricted to a consideration of effects from live load.

Having proportioned one of the multiple arches as a fixed arch, Mr. Janni proceeds to draw influence lines for moments at the kern points for:

(*a*) This arch considered as fixed-ended;
(*b*) This arch considered as the middle arch of three, the flanking arches being fixed at their outer ends, the load, however, being on the central arch only; and
(*c*) The hypothetical flanking arches, the load again being on the central arch only.

He then states that "from the results obtained in this investigation"—that is, from these influence lines—"the following conclusions seem to be justified." These conclusions are, apparently, that:

1.—Professor Guidi's simplification of considering only three arches at a time, and neglecting what lies beyond, is interpreted to mean considering the outer ends of the flanking arches as fixed-ended;
2.—That, if the arches in the series, "no matter how many", satisfy an analysis based on this assumption and on "the conditions of loading as shown in this paper", they will be safe.

The author's phrase "conditions of loading as shown" is very obscure. The influence lines on Plate IV are described first as being for the central arch,

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A_2, and, later, as being for the side arch, A_1. If an influence line for the crown moment on A_2 be drawn for the system as shown and if then an influence line for the crown moment on A_1 or A_3 be drawn for the same system, these lines will not be alike. Subsequently, the author separately discusses the design for the main arch and for the flanking arch; that is, he computes different crown moments for two identical adjacent spans of a series—an anomalous condition. The numerical discrepancy is not great, but the obfuscation of Plate IV is complete. The diagram is especially confusing to any one who thinks of an influence line as a deflected load line. Perhaps the lines may be clarified by transposing the parts of the diagrams under A_1 and A_3, considering the whole line as applying to A_2, and confining the discussion to live load stresses in the central arch. Presumably the author intends that integration under these lines will give maximum kern moments for live load, and thus will solve the problem "in a manner which, in the present state of knowledge, represents the most complete solution that can be desired."

The writer thinks that Professor Guidi's assumption, as stated in the paper, does not necessarily involve fixed-endedness for the flanking arches; that, thus interpreted, it is incorrect, and is dangerous as regards live load stresses in the arch and, in the particular case treated, is on its face highly improbable; and that the acceptance or rejection of "split-load" conditions vitally affects the analysis of continuous arches. Furthermore, he finds in this paper no evidence supporting the assumptions made by its author.

The Ellipse of Elasticity.—Except for the material referred to, the paper is devoted entirely to an exposition of the so-called "theory" of the ellipse of elasticity, the material being much the same as that presented to American engineers by the author on several other occasions. That this method has not become popular with American structural engineers is doubtless explained in part by the brevity of American college courses in Analytic Geometry—college graduates know little about the properties of poles, polars, and anti-poles and are, perhaps, none the worse for the deficiency. As here presented, the method appears as a highly ornamental and somewhat expensive but not especially useful tool for a structural engineer's already crowded workshop. Although probably historically connected with conceptions of the neutral point, of elastic weights, their statical moments and products of inertia, and with the development of familiar properties of equilibrium polygons and their use in determining first and second moments, it is not essential to the application of these conceptions and properties. The use of the ellipse in solving a fixed arch seems to present no advantages; and, in the study of continuous arches, it is somewhat inferior to algebraic analysis in flexibility.

The graphical treatment of moments and products of inertia by means of the ellipse of inertia presents some interesting constructions for studying elastic distortions, especially in continuous girders and arches, just as it does for analyzing stresses in unsymmetrical columns. Its pictorial character appeals to some students, while others consider that its demonstrations require the acquisition of an elaborate paraphernalia for which they can ill afford the time. The writer feels that the beauty of this construction has been some-

what marred in presentation by the author's excursion into a discussion of the properties of equilibrium polygons and by digressions such as the proof of the reciprocal relations, where the use of the ellipse is sadly at a disadvantage compared with the direct use of the principle of virtual work. The writer, by the way, wonders at the author's use of the term "virtual work" in Paragraph 13 and Paragraph 19 of the Appendix, where the work seems very real.

The author states that the application of the ellipse to trusses is simpler than to a solid rib and that the latter application depends on the former. This is not clear; indeed the discussion of trusses in this paper seems entirely irrelevant. In this connection the author states that "in most engineering problems it is not necessary to take into consideration the deformations of the web members of a system". In the computation of deflections, the neglect of the web will frequently result in errors of 20% or more. In all important statically indeterminate trusses, except perhaps swing bridges, the final analysis should always include the effect of the web; for approximate analyses, solutions by elastic weights are, in general, not so accurate, so simple, nor so expeditious as by the theorems of beam analysis. The omission of the effect of the web members by Mr. Janni was the main point of reference to the ellipse of elasticity by George F. Swain, Past-President, Am. Soc. C. E., cited in the footnote on page 1160.

Algebraic Analysis.—General Formulas.—The writer has studied by algebraic methods the properties of the arch system shown, following as nearly as possible the author's method of reasoning, although he does not consider this the simplest method of attack.

Consider any arch and pier, *A B C*, as shown in Fig. 24. It is desired to determine the elastic properties of the system for forces and moments acting at *B*. These elastic properties will be defined as;

1.—The co-ordinates of the neutral point, which may be defined as that point which, if rigidly attached to *B*, will suffer no rotational displacement due to forces acting through it. (This is the center of the author's ellipse of inertia.)

2.—The elastic weight, or the rotation of the neutral point due to a unit applied moment. (This is the same as the author's elastic weight.)

3.—The moment of inertia about a vertical axis through the neutral point, the moment of inertia about a horizontal axis through the neutral point, and the product of inertia about horizontal and vertical axes through the neutral point. These represent, successively, (*a*) the vertical displacement of the neutral point due to a unit vertical force acting through it; (*b*) the horizontal displacement due to a unit horizontal force; and (*c*) the vertical displacement due to a unit horizontal force, or *vice versa*. (These correspond to radii of the author's ellipse.)

Assume the corresponding elastic properties of the arch, *A B*, and of the pier, *B C*, to be known. Let the total elastic weight equal W; the moment of inertia about a vertical axis, I; the moment of inertia about a horizontal axis, J; and the product of inertia about vertical and horizontal axes, Z, subscripts being used to designate whether for the arch (by a), for the pier (by p),

or for the combined system (by s). Otherwise, the nomenclature is fully explained by Fig. 24.

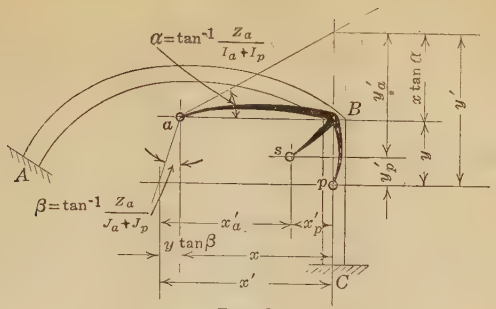

FIG. 24.

The three neutral points, a for the arch, p for the pier, and s for the system, are connected to B by imaginary inelastic bracket arms and hence constitute a rigid system. Apply at s a unit horizontal force (Fig. 25(a)). As this is a neutral point, there will be no rotation of s and, therefore, none at a or p. Hence, $M_a = M_p = 0$. $H = 1$, therefore, is balanced by H_a, H_p, and a V-couple.

Vertical displacement of a, due to $V = d\,V_v^a = +\,V\,I_a$
" " " a, " " $H_a = d\,V_{H_a}{}^a = -\,H_a\,Z_a$
" " " a (total) $= V\,I_a - H_a\,Z_a$
" " " p, due to $V = d\,V_v^p = -\,V\,I_p$
" " " p, " " $H_p = V_{H_p}{}^p = H_p\,Z_p = 0$
" " " p (total) $= -\,V\,I_p$

As there is no rotation of s, the vertical displacements of a and p are the same, and,

$$V\,(I_a + I_p) - H_a\,Z_a = 0$$

$$\frac{V}{H_a} = \frac{Z_a}{I_a + I_p}$$

Hence, the resisting force at a acts along a line making with the horizontal an angle, $\alpha = \tan^{-1}\dfrac{Z_a}{I_a + I_p}$, and equals $\dfrac{H_a}{\cos\alpha}$. Also, the horizontal movements of the two points are the same and, following the system of designating displacements indicated, $d\,H_s = d\,H_p = H_p\,J_p = d\,H_a = \dfrac{H_a}{\cos\alpha}$ (product of inertia for the arch about this inclined axis and about the horizontal axis),

$$= H_a\left(J_a - \frac{Z_a{}^2}{I_a + I_p}\right)$$

Let this new quantity,

$$J_a - \frac{Z_a{}^2}{I_a + I_p} = J_a'$$

Then,

$$\frac{H_a}{H_p} = \frac{J_a'}{J_p}$$

By statics,

$$H_a + H_p = 1$$

Hence,

$$d\,H_s = J_s = \frac{J_a' J_p}{J_a + J_p}$$

$$\frac{1}{J_s} = \frac{1}{J_a'} + \frac{1}{J_p}$$

Again, by statics,

$$\frac{y_a'}{y_p'} = \frac{H_p}{H_a} = \frac{\frac{1}{J_p}}{\frac{1}{J_a'}}$$

$$y_a' + y_p' = y'$$

$$y_p' = \frac{J_s}{J_a'}\, y'$$

Also,

$$Z_x = V I_p = H_a \tan \alpha\, I_p = I_p \frac{J_s}{J_a'} \tan \alpha$$

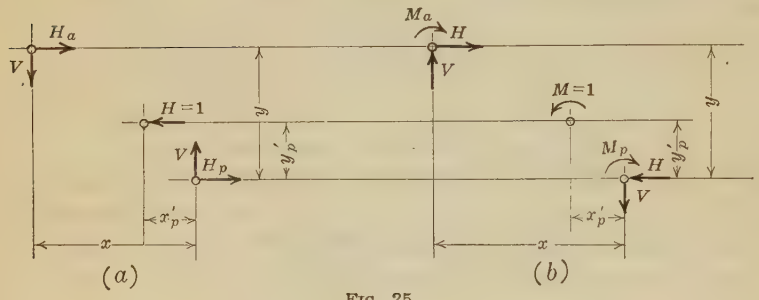

FIG. 25.

By an exactly similar method of reasoning:

$$\frac{1}{I_s} = \frac{1}{I_p} + \frac{1}{I_a'}$$

Where,

$$I_a' = I_a - \frac{Z_a^2}{J_a + J_p}$$

$$x_p' = \frac{I_s}{I_a'}\, x'$$

Where,

$$x' = x + y \frac{Z_a}{J_a + J_p}$$

Now, apply at s a unit moment (Fig. 25 (b)). This will produce at s a rotation, W_s, but no translation. It will be resisted by four couples, M_a, M_p, H_y, and V_x.

As the three points rotate together,

$$M_a W_a = M_p W_p = W_s$$

$$M_a = \frac{W_s}{W_a};\ M_p = \frac{W_s}{W_p}$$

As at x there is rotation only,

$$d H_p = H J_p = y_p' W_s$$

$$H = \frac{y_p'}{J_p} W_s$$

$$d V_p = V I_p = x_p' W_s$$

$$V = \frac{x_p'}{I_p} W_s$$

By statics,

$$M_a + M_p + H_y + V_x = 1$$

$$\frac{W_s}{W_a} + \frac{W_s}{W_p} + \frac{y\, y_p'}{J_p} + \frac{x\, x'}{I_p} = 1$$

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_p} + \frac{x\, x_p'}{I_p} + \frac{y\, y_p'}{J_p}$$

and the resisting couples are proportional to the four quantities on the right of the equation.

These reduction formulas are perfectly general, since it is always possible to choose the axes of x and y so that they are the principal axes of one of the branches, for which $Z = 0$.

Since the expression for W_s when I_p or $J_p = 0$, is indeterminate, it is permissible to write:

$$\frac{y_p'}{J_p} = \frac{y'}{J_a + J_p}$$

$$\frac{x_p'}{I_p} = \frac{x'}{I_a' + I_p}$$

$$\frac{x\, x_p'}{I_p} + \frac{y\, y_p'}{J_p} = \frac{x\, x'}{I_a' + I_p} + \frac{y\, y'}{J_a' + J_p}$$

$$= \frac{x\left(x + y\,\dfrac{Z_a}{J_a + J_p}\right)}{I_a + I_p - \dfrac{Z_a^2}{J_a + J_p}} + \frac{y\left(y + x\,\dfrac{Z_a}{I_a + I_p}\right)}{J_a + J_p - \dfrac{Z_a^2}{I_a + I_p}}$$

$$= \frac{x^2}{I_a + I_p} + \frac{(y')^2}{J_a' + J_p} = \frac{(x')^2}{I_a' + I_p} + \frac{y^2}{J_a + J_p}$$

If $I_p = 0$, as assumed subsequently,

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_p} + \frac{x^2}{I_a} + \frac{(y')^2}{J_a' + J_p}$$

Although it is not necessary to do so, it is convenient to take $I_p = 0$. This is equivalent to neglecting rib-shortening (direct compression) in the pier.

Data are presented later to show the effect of this assumption in the case discussed by the author.

Summarizing, when $I_p = 0$,

$$y' = y + x\frac{Z_a}{I_a}$$

$$J_a' = J_a - \frac{Z_a^2}{I_a}$$

$$\frac{1}{J_s} = \frac{1}{J_p} + \frac{1}{J_a'} = \frac{J_a' J_p}{J_a' + J_p}$$

$$I_s = 0 = Z_s$$

$$y_p' = \frac{J_s}{J_a'} y'$$

$$\frac{1}{W_s} = \frac{1}{W_a} + \frac{1}{W_b} + \frac{x^2}{I_a} + \frac{(y')^2}{J'_a + J_p}$$

Critical Study of the Author's Structure.—The paper gives the following data for the pier and the fixed-ended arch:

$$W_a = 373.5$$
$$I_a = \rho_v^2 W_a = (12.90)^2 \times 373.5 = 62\,100$$
$$J_a = \rho_h^2 W_a = (1.30)^2 \times 373.5 = 630$$
$$Z_a = 0 \qquad x = 25.67 \text{ ft.} \qquad Y = 18.87 \text{ ft.}$$
$$W_p = 74.6$$
$$I_p = 74.6 \times (0.50)^2 = 18.7$$
$$J_p = 74.6 \times (10.90)^2 = 8\,900$$
$$Z_p = 0$$

As the arch is symmetrical, $Z_a = 0$; $x = x'$; and $y = y'$. Therefore,

$$\frac{1}{W_s} = \frac{1}{373.5} + \frac{1}{74.6} + \frac{(25.67)^2}{62\,100 + 18.87} + \frac{(18.87)^2}{8\,900 + 630} = \frac{1}{15.65}$$

$$J_s = \frac{8\,900}{8\,900 + 630} 630 = 588\,; \quad I_s = \frac{62\,100}{62\,100 + 18.9} 18.9 = 18.9$$

$$y_p = \frac{588}{630} 18.87 = 17.6\,; \quad x_p = \frac{I_p}{I_a + I_p} x = \frac{18.87}{62\,119} 25.67 = 0.008$$

The author gives:

$$W_s = 15.55 \text{ (average of 15.50 and 15.60)}$$
$$J_s = 15.55 \times 38.10 = 593$$
$$I_s = 15.55 \times 1.19 = 18.50$$
$$y_p = 17.596; \qquad x_p = 0.007$$

This equivalent pier may now be combined with the next arch (see Fig. 26) disregarding I_p which, as is obvious from the previous computation, has a negligible effect.

The centroid of W_a and W_s lies:

$$\frac{15.65}{373.5 + 15.65} 25.67 = 1.03 \text{ ft. to the right of } W_a$$

and

$$\frac{15.65}{373.5 + 15.65} 1.27 = 0.05 \text{ ft. below } W_a$$

$$W_t = W_a + W_s = 373.5 + 15.65 = 389.2$$

$$I_t = I_a + \frac{W_a W_s}{W_a + W_s} (25.67)^2 = 62\,100 + 9\,900 = 72\,000$$

$$J_t = J_a + J_s + \frac{W_a W_s}{W_a + W_s} (1.27)^2 = 630 + 588 + 24 = 1\,242$$

$$Z_t = \frac{W_a W_s}{W_a + W_s} 25.67 \times 1.27 = 490$$

$$x = 25.67 + 1.03 = 26.70\,; \quad y = 18.87 - 0.05 = 18.82$$

These are the values of the new arch which is to be combined with the pier, as follows:

$$y' = 18.82 + 26.70 \frac{490}{72\,000} = 18.99$$

$$J_a' = 1\,242 - \frac{(490)^2}{72\,000} = 1\,242 - 3 = 1\,239$$

$$\frac{1}{W_s} = \frac{1}{389.2} + \frac{1}{74.6} + \frac{(26.70)^2}{72\,000} + \frac{(18.99)^2}{8\,900 + 1\,239} = \frac{1}{16.28}$$

$$J_s = \frac{8\,900}{8\,900 + 1\,239} 1\,239 = 1\,085; \; y_p = \frac{1\,085}{1\,239} 1\,899 = 16.65$$

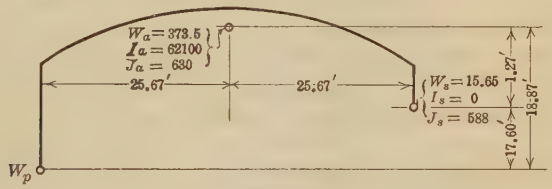

FIG. 26.

Study of Crown Thrust.—A comparison of the two sets of computations just made, shows that if the series is taken as five arches instead of three, very different results obtain. A further study will show what effect this will have on the horizontal thrust at the crown of the middle arch due to a unit load at this point. A unit horizontal force at a (Fig. 27) will produce a vertical deflection of $b = i_h J_a$, where i_h is the influence ordinate at b for the horizontal thrust on the fixed arch. Also, a unit moment at a will produce a vertical deflection at b equal to $i_m W_a e_y$, in which, i_m is the influence ordinate at b for the moment at the neutral point of the fixed arch. In Table 3, the author gives $i_h = 2.507$. The influence ordinate for the moment at b is given as:

$$\frac{1.980 + 2.857}{2} = 2.418 \text{ (Table 3)}$$

and the distance, $a\ b$, scales approximately 1.5 ft. on Plate III. Hence,

$$i_h J_a = 2.507 \times 630 = 1\,580$$

$$i_m W_a e_y = (2.418 + 1.5 \times 2.507)\, 373.5\, e_y = 2\,310\, e_y$$

The vertical deflection of Point b of the arch with elastic piers, due to a unit horizontal force at O, is, therefore,

$$i_h J_a - i_m W_a e_y + W_x x d$$

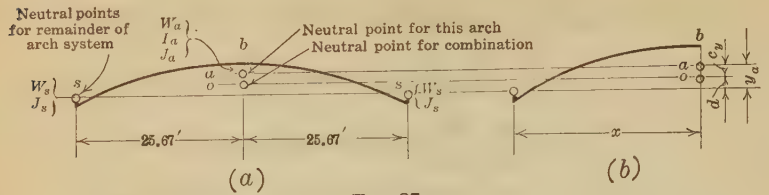

FIG. 27.

The new influence ordinate for H is the quotient of this value and the moment of inertia about the horizontal axis through O of the combination, which will be called J_O.

Applying this to the combinations found previously, if,

$$W_s = 15.65,\ J_s = 588,\ y_a = 1.27$$

$$e_y = \frac{2 \times 15.65}{373.5 + 2 \times 15.65} 1.27 = 0.10$$

$$d = 1.27 - 0.10 = 1.17$$

$$i_h J_a + W_s x d - i_m W_a e_y = 1\,580 + 15.65 \times 25.67 \times 1.17 - 2\,310 \times 0.10 = 1\,825$$

$$J_O = J_a + 2 J_s + \frac{2 W_s W_a}{W_a + 2 W_s} (1.27)^2 = 630 + 1\,176 + 47 = 1\,853$$

$$i_h' = \frac{1\,825}{1\,853} = 0.99$$

In Table 7 the author gives $i_h' = 1.009$.

If, however, a series of five arches is taken:

$$W_s = 16.28;\ J_s = 1\,085;\ y_a = 18.85 - 16.65 = 2.22$$

$$e_y = \frac{2 \times 16.28}{373.5 + 2 \times 16.28} 2.22 = 0.18$$

$$d = 2.22 - 0.18 = 2.04$$

$$i_h J_a + W_s x d - i_m W_a e_y = 1\,580 + 16.28 \times 25.67 \times 2.04 - 2\,310 \times 0.18 = 2\,057$$

$$J_O = J_a + 2 J_s + \frac{2 W_a W_s}{W_a + 2 W_s} (2.22)^2 = 2\,947$$

$$i_h' = \frac{2\,057}{2\,947} 0.697$$

The writer has extended this investigation, including, successively, three, five, seven, nine, etc., arches in the series (the ends of the series being fixed), and has determined the effect on the influence ordinate for H at the center of the middle arch. The data are given in Table 9.

TABLE 9.—VARIATION IN CERTAIN PROPERTIES OF THE MIDDLE ARCH ACCORDING TO THE NUMBER OF ARCHES IN THE SYSTEM.

Number of arches considered.	Rotational resistance of remainder of series, W_s.	Horizontal resistance of remainder of series, J_s.	Location of center of rotation of remainder of series, y_a, in feet.	Influence Ordinate for Thrust at Crown of Middle Arch Due to Load There, i_h		
				True value.	Author's value.	Author's error, percentage.
1	.....	.....	....	.....	2.507	....
3	15.65	588	1.27	0.99	1.009	0
5	16.28	1 085	2.22	0.697	1.009	45
7	16.65	1 489	2.92	0.560	1.009	80
9	17.00	1 815	3.57	0.517	1.009	95
11	17.25	2 022	3.99	0.465	1.009	120
13	17.38	2 170	4.27	0.450	1.009	124

It is evident that the properties of the remainder of the series of arches on each side are represented by convergent series which, in the case selected by the author, converge slowly. This is due to the extremely slender piers used in the example chosen. In the extreme case of piers offering only vertical resistance (horizontal resistance, nil), the value of J_s, which measures the horizontal movement of one end of a series of arches due to horizontal loading is directly proportional to the number of arches in the series. The author, however, assumes that, in cases approaching this, the horizontal movement is independent of the number of arches in the series.

The Author's Structure not Representative.—The example taken by the author seems unfortunate in that the arches are so flat and the piers offer so little resistance to horizontal movement that the case approaches that of a continuous girder, as far as the stresses in the arch are concerned. By summing the influence ordinates, as contained in the paper, for a load, W, uniformly distributed over the middle arch: The crown thrust for an arch on piers $= 0.653\ W$. Taking the scaled distance, 1.5 ft. from the crown to the neutral point, the moment at the crown, due to the thrust (if on piers) $= 1.5 \times 0.653\ W = 0.979\ W$. Taking a live load of 150 lb. per sq. ft. ($W = 50 \times 150$ lb. per ft. of width), the reduction in crown moment for a 1-ft. strip of arch ring (assuming a barrel arch) equals:

$$50 \times 150 \times 0.979 = 7\ 350 \text{ ft-lb.}$$

The temperature moment due to a change of 50° equals:

$$\frac{50° \times 0.000006 \times 50 \text{ ft.} \times 1\ 500\ 000 \times 144}{630} 1.5 = 7\ 700 \text{ ft-lb.}$$

By fixing against horizontal movement the ends of these arches, the loss in temperature stresses is about as much as the gain—even on the author's assumptions—in live load stresses. Any economy would be in the dead load stresses; to erect such a series if it were many spans in length without sacrificing this economy by increasing the size of pier to provide for unbalanced erection thrusts would present a difficult problem. Only strong æsthetic considerations would apparently justify building this series as shown, and even then

the average structural engineer would probably design the arch as a continuous girder.

Complete Analysis Practicable.—Finally, the writer believes that a correct and practically complete analysis of a mutiple-arch system is quite feasible and that the amount of labor involved is not unreasonable in the case of an important structure; the correct and economical design of the system is another matter. The writer heartily concurs in the author's statement that:

"The designer should investigate the behavior of a multiple-arch system in every case where the importance of the spans, the comparative slenderness of the piers, and the conditions of the foundation are such as to warrant a thorough investigation."

He would like to emphasize strongly the last two words.

Harold B. Hammill,* Assoc. M. Am. Soc. C. E. (by letter).—American engineers are indeed indebted to Mr. Janni for presenting in English this improved method of analyzing an arch, namely, by the ellipse of elasticity. To the best of the writer's knowledge his presentation† of the method was the first published in English.

In association with John B. Leonard, M. Am. Soc. C. E., the writer has been using this method for several years and finds it both quick and accurate. Some engineers object to it as a graphical determination for stresses, claiming that it is, for this reason, less accurate than an analytical one, but the writer has found that analytical and graphical computations by the usual elastic theory will check within 2 per cent.

Of interest to engineers is the greater accuracy obtained by this method. At best, the equations expressing the elastic relations of an arch ring are based on several assumptions. By the determination of the "centers of rotation" of segments of the arch ring, the number of assumptions is reduced by one and the problem is brought one step nearer an exact analysis. In some cases, stresses determined in this manner exceed those obtained by the generally accepted elastic theory by percentages that are greatly in excess of those due to the use of graphics. The writer has found that some arches, considered as fixed-ended, show the stresses determined by the ellipse analysis to be as much as 20% greater than by the usual elastic theory.

This variation is influenced by the ratio of rise to span length as well as by the dimensions of the rib itself. As the ratio of rise to span length increases, the difference in stress decreases; for an arch with a very low ratio, it would be considerable.

The inclusion of the elastic piers in the analysis of a series of arches influences the stresses in proportion to the elasticity of the pier. For any unbalanced loads the stresses in an arch of a series are always greater than those determined from the analysis of the arch alone.

Fig. 28 (*a*) shows the outline and general dimensions of a structure of three equal spans for which the stresses have been determined by three analyses, the results of which are given in Table 10.

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* San Francisco, Calif.

† *Journal*, West. Soc. of Engrs., May, 1913.

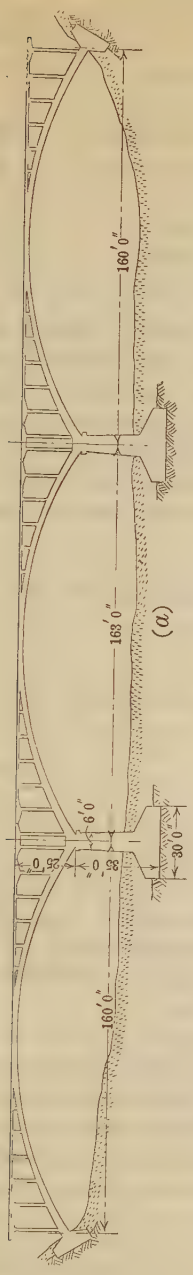


FIG. 28.—ELEVATIONS AND DIMENSIONS OF TRIPLE ARCH BRIDGES.

TABLE 10.—Maximum Compressive Stresses, in Pounds per Square Inch, Arch of Fig. 28 (*a*).

	Single Fixed-Ended Arch.		Three-Span Unit with Piers.
	Ordinary elastic method.	Author's method.	Author's method.
At crown	660	680	797
At point near crown	750	764	828
At haunch	825	960	1 025

Fig. 28 (*b*) shows the outline and general dimensions of a structure designed for a stream in low-lying country. The fact that the distance between the roadway level and high-water elevation was limited, made it desirable to proportion the spans such that the side spans were shorter than the central one. The ratio of rise to span length was kept approximately constant for all spans. In Table 11 are recorded the stresses in the central span computed by the usual elastic method for a fixed-ended arch and by the author's method, including piers.

TABLE 11.—Maximum Compressive Stresses, in Pounds per Square Inch, Arch of Fig. 28 (*b*).

	Ordinary elastic method.	Author's method.
Point near crown	692	930
Point near haunch	614	928

A designer controlled by pre-established maximum stresses would be considerably in error for such a structure as that shown in Fig. 28 (*b*), if he accepted the results of the usual fixed-ended rib analysis as correct. The value for the maximum foundation pressure would be too great, as the adjacent arch takes some of the unbalanced thrust and the value for the maximum compression in the rib would be too low.

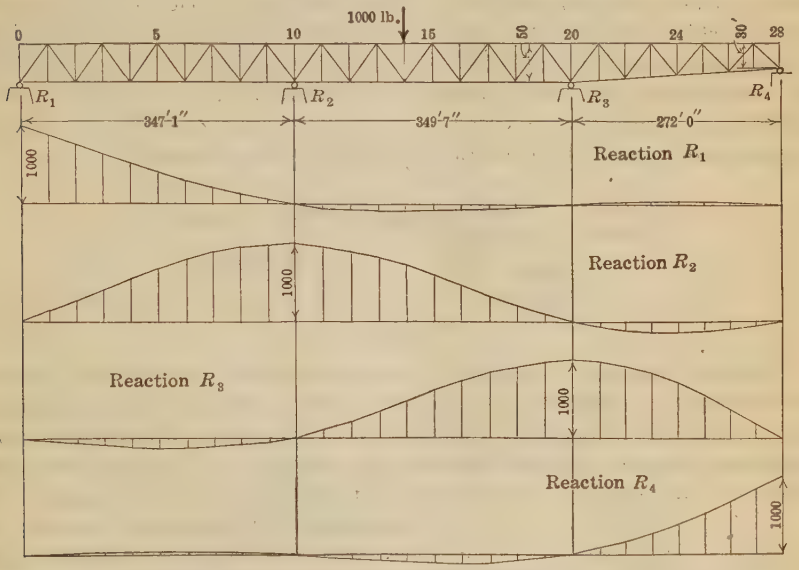
From experience with bridges of more than three spans the writer can concur with Mr. Janni in his conclusion that the unit of three spans is sufficient for determining the maximum stresses. The relative pier dimensions of those structures which have been investigated, have always been much larger than those of the author's example, due to considerations of stiffness. Experience shows that the difference in stress computed by a three-span unit *versus* a five-span unit would not exceed 2 per cent.

George E. Beggs,* M. Am. Soc. C. E.—The speaker admires the scholarly mathematical treatment of the multiple arch as made by the author, and has

* Associate Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

great confidence in the accuracy of his theoretical analysis, because it checks with remarkable precision the values obtained by the speaker in a laboratory experiment made on a model of this triple arch.

The method of solving problems of indeterminate structures by the use of elastic models of proportionate stiffness is based on a much used and well-known principle—Maxwell's Theorem of Reciprocal Deflections. The idea of using this theorem to obtain influence lines by observing the microscopic movements of elastic structural models was suggested by a remarkable generalization by D. B. Steinman, M. Am. Soc. C. E., that "every influence line is a deflection diagram".* The truth of this statement was brought home to the speaker in 1916 when drawing the influence lines for the reactions of the Bessemer and Lake Erie Bridge near Pittsburgh, Pa., a continuous structure resting on four supports. (See Fig. 29.) It was very apparent that these influence lines were deflection diagrams produced in some definite manner.



INFLUENCE LINES CONTINUOUS TRUSS ALLEGHENY RIVER BRIDGE
BESSEMER AND LAKE ERIE R. R.
FIG. 29.

A complete understanding of this statement is helpful to the engineer who must devise analytical or graphical methods for the determination of influence lines for stresses and deflections of both determinate and indeterminate structures. To the speaker occurred the idea that, if "every influence line is a deflection diagram", it might be possible to obtain the values for an influence line by observing certain deflections of an elastic model. This led to the development of a method for which the mathematics is as simple as that of the see-saw. In fact, the equation between load, reaction, and deflections is the same for an indeterminate structure as for the determinate see-saw.

* *Engineering Record*, November 25, 1916, p. 648; also, *Engineering Record*, May 20, 1916, p. 680.

(Fig. 30.) The see-saw is supported at a point other than the center and weighted with a known load at the left end. By measuring with precision the distance that the left end is depressed when the right end is moved up a known amount, the exact reaction that must be applied to the right end to produce equilibrium is given by the equation:

$$\text{Reaction} = (\text{Load})\frac{\text{Deflection 2}}{\text{Deflection 1}} = P\frac{d_2}{d_1}$$

Evidently, this ratio of the two deflections may also be found from an unloaded model of the see-saw by subjecting the model to a small known deflection at the reaction point and measuring the deflection at the load point.

In dealing with a structure in which the lever arms of the known and unknown forces are easily determined, as in the foregoing example, there is no advantage in using the method of deflections, nor of using models to obtain the ratio of the deflections; but with an indeterminate structure there are usually no evident lever arms that relate the known loads to the unknown reactions which they produce. The mathematical solution must then be based on the elastic theory of structures, of which the calculation of certain deflections is an important part. Such a mathematical analysis may often be avoided if a model of the structure made of elastic material, be deflected at certain points and the deflections at other points measured.

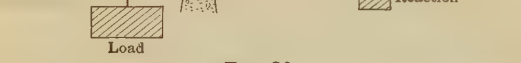


FIG. 30.

When the component of a reaction of a complicated indeterminate structure is to be found, a model of the structure is cut, preferably from celluloid or heavy cardboard of constant thickness, with the width of members proportioned according to the cube root of the moment of inertia of the actual members. This model is placed on a flat drawing-board and supported by bearing balls to minimize friction. At the free ends or supports of the model, small devices called "deformeter gauges" (see Fig. 31) are attached. The deformeter gauge consists of two parallel metal bars 4 in. long and ½ in. wide, arranged so that one bar is fastened to the drawing-board by two screws, and the other allowed to move freely. The free end of the model structure is fastened to the movable metal bar either by a rigid clamp or by a hinge pin, depending on the type of support assumed. At each side of the center line of the two metal gauge bars are opposing V-shaped notches made to receive

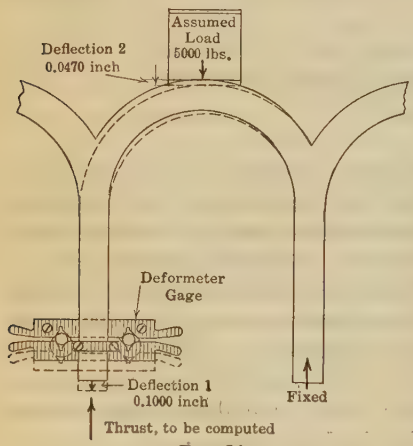


FIG. 31.

gauge plugs. The plugs in the notches when no deformation of the model is produced are called "normal plugs". If the normal plugs are removed and two equal sized plugs of larger diameter are substituted, the movable bar will be caused to deflect parallel to itself. In Fig. 31, for example, this downward deflection, 1, is shown as 0.1000 in. Suppose that with a micrometer microscope, Deflection 2 measures 0.0470 in., and that the assumed load at Deflection 2 is 5 000 lb.—it follows from Maxwell's theorem that the vertical component of the reaction at the left-hand column is given by:

$$\text{Thrust} = (\text{Load})\frac{\text{Deflection 2}}{\text{Deflection 1}} = (5\ 000)\frac{0.0470\text{ in.}}{0.1000\text{ in.}} = 2\ 350\text{ lb.}$$

This equation is identical with that used to solve the "see-saw" problem. Deflection 1 is arbitrarily chosen, Deflection 2 is measured with a microscope, the load is assumed, and the component of the reaction follows by multiplying the assumed load by the ratio of the two deflections.

Reference to Fig. 32 will show clearly the method of using laboratory apparatus to solve an unsymmetrical multiple-arch system of two spans with fixed supports, *X*, *Y*, *Z*, subjected to any inclined load, *P*, at a point, *D*. Let

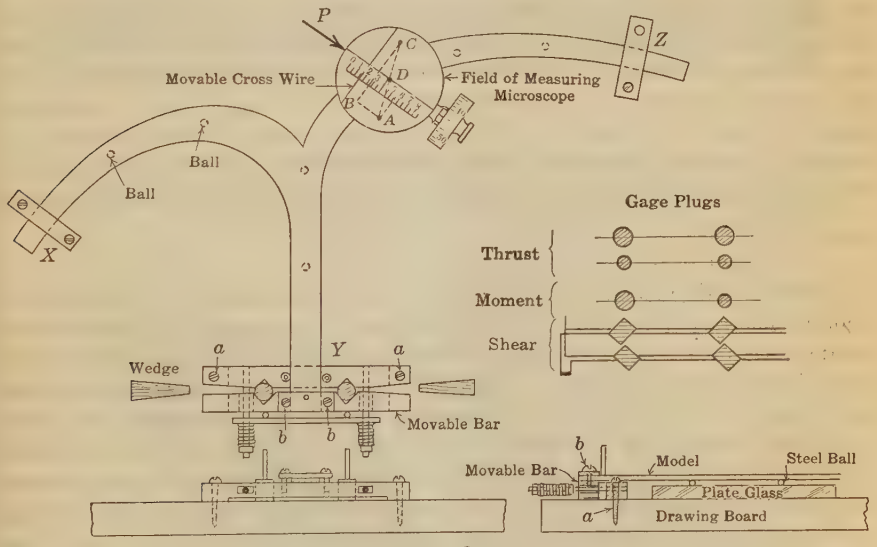


FIG. 32.

it be desired to find by model deflections the value of the thrust, shear, and moment at the support, *Y*, caused by the assumed load, *P*. The model is attached to a flat drawing-board by clamps, *X* and *Z*, and by a deformeter gauge at *Y*, and supported by steel bearing balls which roll on plate glass. The fixed bar of the gauge is fastened to the board by two screws, *a*; the model is fastened to the movable bar of the gauge by a clamp and screws, *b*. The movable bar is thrust by springs against the two "normal plugs" in the V-notches when there is no stress in the model. By means of two wedges pushed into the tapered space between the movable and fixed bars, it is pos-

sible to increase or decrease the distance between the V-notches after the removal of the "normal plugs", or to cause the movable bar to move to the left or right. It is possible, therefore, to fit into the V-notches various sized gauge plugs as shown, and so produce at will a predetermined amount of pure vertical movement, pure horizontal movement, or pure rotative movement of the support, Y, of the model.

Over the assumed load point, D, of the model is set up a filar micrometer microscope with the axis of its screw oriented in the direction of the assumed load, P. This microscope can then be used to measure accurately the component, BA, of the deflection, CA, this component always being taken in the same direction as P.

In Fig. 33 is shown a box of six standard type deformeter gauges, including the necessary normal, moment, shear, and thrust plugs. Fig. 34 shows a micrometer microscope in use to determine influence values for the moment, shear, and thrust at the support of a hingeless arch. Fig. 46 is a simple equipment used at the University of Illinois by F. E. Richart, Assoc. M. Am. Soc. C. E., in the study of the effect of knee-brackets on the strength of portals. Figs. 35, 36 and 37 indicate how certain deflections of a model of Mr. Janni's triple arch may be used to evaluate the components of the reaction at one of the supports. Referring in particular to Fig. 35, which illustrates the manipulation of the model to give the vertical component, V, of the reaction as caused by any assumed load, P, by means of a deformeter gauge, the foot of the model is deflected through a small known distance, d_1, and the micrometer microscope reads the component, d_2, of the deflection of the assumed point of loading. It follows immediately from Maxwell's theorem

that $V = P\,\frac{d_2}{d_1}$.

By an exactly analogous method, the value of the horizontal component, H, and the moment, M, due to any assumed load, P, is found by multiplying P by the ratio of one observed model deflection, d_2, to one known support deflection, d_1, as clearly indicated in Figs. 36 and 37, respectively. The diagrams also show how the effect of inclined loads is determined. Operating the model of the multiple arch (Figs. 35, 36, and 37), gives a feeling of the manner in which the applied loads are related to the stresses at the support, and the more the experience with such models, the closer will be the guess of the amount of thrust, shear, and moment that is produced by a given applied load. The judgment as to stress distribution will be so improved that errors in the results obtained by tedious and disheartening solutions of numerous simultaneous equations will be apparent immediately. Attention is called particularly to the fact that no weights are applied to the model in order to obtain the solution, for the method depends solely on the measurement of certain deflections of the unloaded model, also on the fact that all the reaction components are found by the ratio of certain deflections. There remain, therefore, the unused three equations of static equilibrium, which at the end may be used as a triple check on the results obtained. For mathematical solutions such a simple independent check is not available. The model method is as simple for non-

Fig 33.—Set of Deformeter Gauges.

Fig. 34.—Reading Influence Values for an Arch with Fixed Ends.

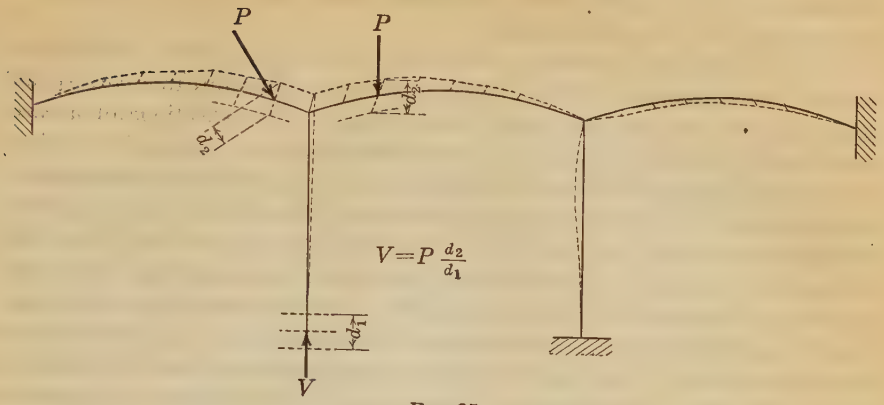


FIG. 35.

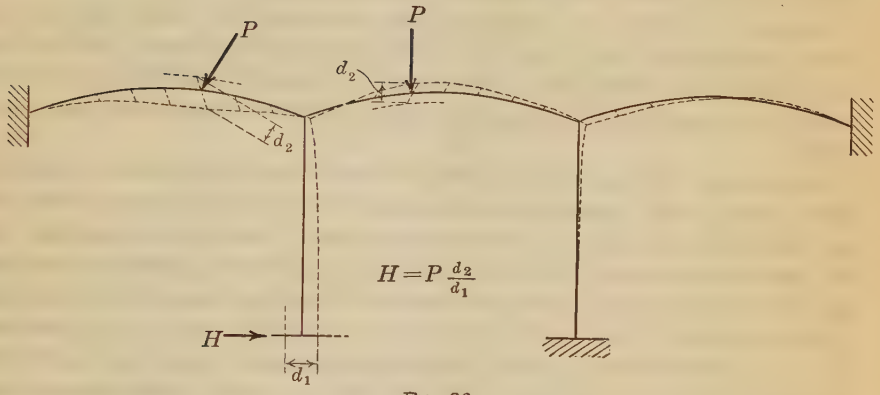


FIG. 36.

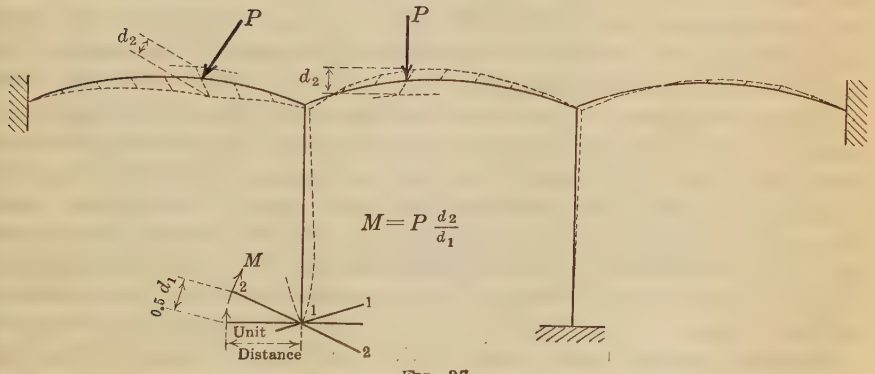


FIG. 37.

symmetrical as for symmetrical plane structures, and as easily applied for inclined loads as for vertical loads.

In Figs. 38 to 43, inclusive, are shown the influence lines for the components of the reactions of Mr. Janni's triple arch. The full line corresponds to values obtained from laboratory observations of the deflections of a cardboard model; the dotted lines below the middle span give the results obtained by Mr. Janni, using the ellipse of inertia method. In general, the reactions at a given support are small for loads at greatest distances from it. Two important exceptions to this are the horizontal thrust, H_A (Fig. 38) and the M_A at the left support (Fig. 40), which illustrate the danger of calculating stresses for the first or left-hand arch on the assumption that the loads on the third span may be neglected; for it is evident from the influence lines that loads in the third span produce both large positive moment and large positive thrust at the left support of the first span. The speaker would hesitate, therefore, to endorse the general rule that a system of more than three arches on elastic piers may be designed safely on the assumption that "three adjacent arches of [such a] system [may be considered] as forming an elastic system by itself, disregarding what is on either side of this selected unit, that is, the outer ends of this three-arch system should be regarded as fixed". Such an assumption is particularly dangerous for an elastic system with very flat arches.

The possible "fading out" of influence lines in remote spans of an elastic system was of such interest that the speaker decided it was worth while to test an elastic arch system having eleven spans of the same proportions as Mr. Janni's triple arch. In Figs. 44 and 45 are shown the influence lines for the horizontal thrust at the left end and for the bending moment at the crown of the sixth arch. It is particularly evident from Fig. 44 that the loads on the spans far removed from the one under discussion must be considered when dealing with a long series of flat arches on elastic piers subjected to unbalanced loading. This influence line shows clearly the cumulative effect on horizontal thrust of applying earth fill to such a system progressively from the right to the left end. The influence line in Fig. 45 indicates the cumulative effect on the negative crown moment that would be produced if the earth fill were applied progressively from each end toward the middle arch.

In order to check further the results obtained by the author and by the speaker, a celluloid model was constructed to the same proportions as the triple arch. By combining values of the reaction components as taken from the influence lines previously obtained with a paper model, the positions, directions, and magnitudes of the four reactions were found for the several cases of loading illustrated in Figs. 47, 48, 49, 50, and 51. The loads appear as weights hung from the arches, while the reactions are completely indicated by white arrows. From Fig. 47 it is apparent that the horizontal thrust is transmitted through several arches until it reaches the support of the right arch with considerable force. The correctness of the position of the reactions was indicated in each case by the fact that the four foundations were sup-

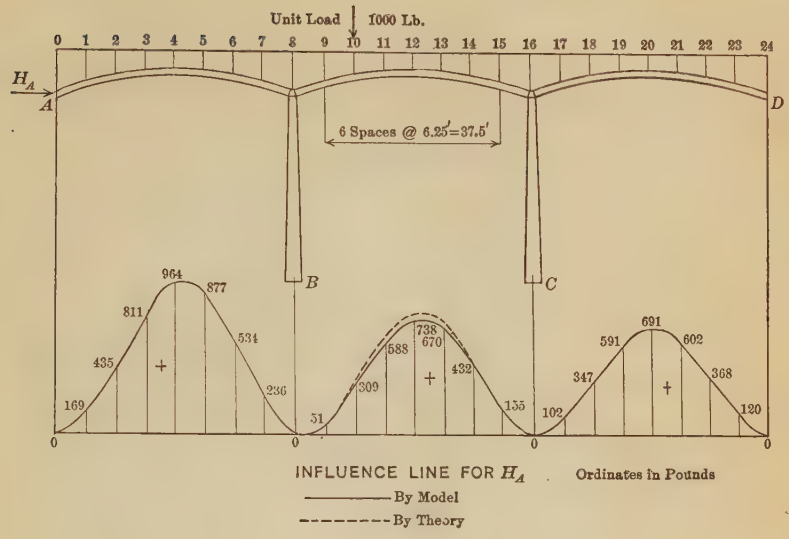

Unit Load 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
H_A
A
D
6 Spaces @ 6.25'=37.5'
B
C
964
877
811
534
435
236
169
738
670
588
432
309
155
51
691
602
591
368
347
120
102
0
INFLUENCE LINE FOR H_A
Ordinates in Pounds
By Model
By Theory

FIG. 38.

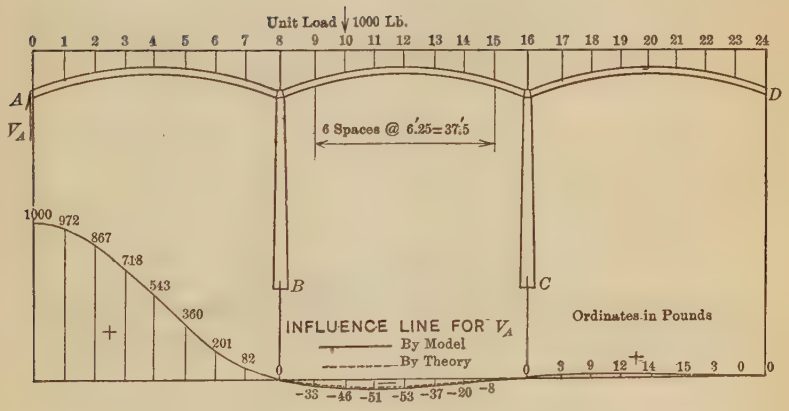

Unit Load 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
A
V_A
D
6 Spaces @ 6.25=37.5
1000
972
867
718
543
360
201
82
B
C
INFLUENCE LINE FOR V_A
By Model
By Theory
Ordinates in Pounds
-33 -46 -51 -53 -37 -20 -8
3 9 12 14 15 3 0

FIG. 39.

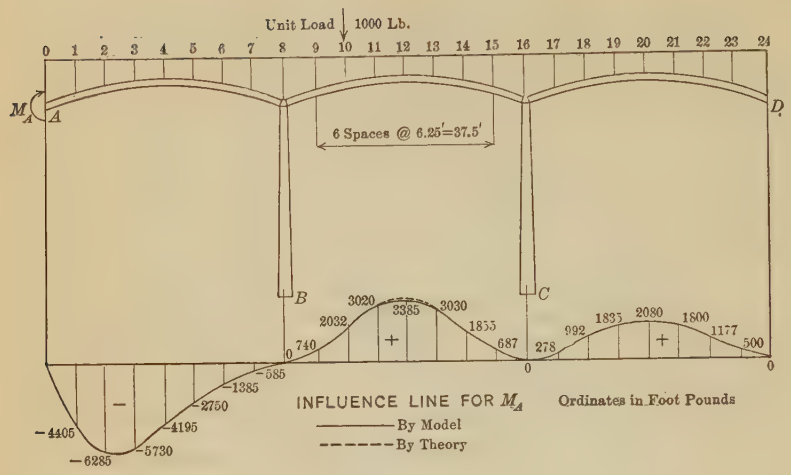

Unit Load ↓ 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
M_A
A
D
6 Spaces @ 6.25′=37.5′
B
C
3020
2385
3030
2032
1855
740
687
0
278
992
1835
2080
1800
1177
500
0
0
−585
−1385
−2750
−4195
−4405
−5730
−6285
INFLUENCE LINE FOR M_A
Ordinates in Foot Pounds
By Model
By Theory

FIG. 40.

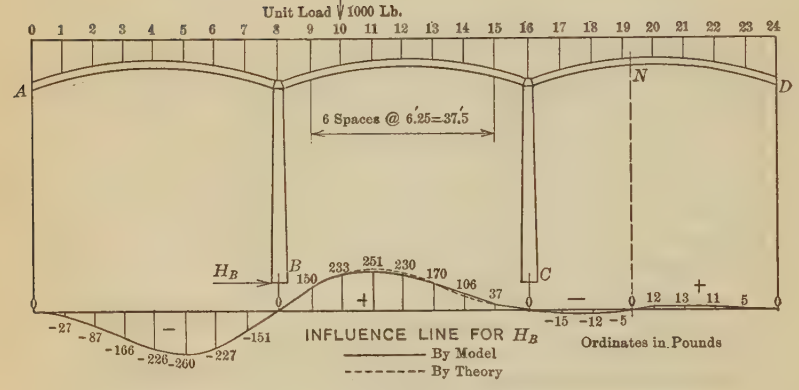

Unit Load ↓ 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
A
N
D
6 Spaces @ 6.25′=37.5′
H_B
B
C
150
233
251
230
170
106
37
0
12
13
11
5
−15
−12
−5
−27
−87
−166
−226
−260
−227
−151
INFLUENCE LINE FOR H_B
Ordinates in Pounds
By Model
By Theory

FIG. 41.

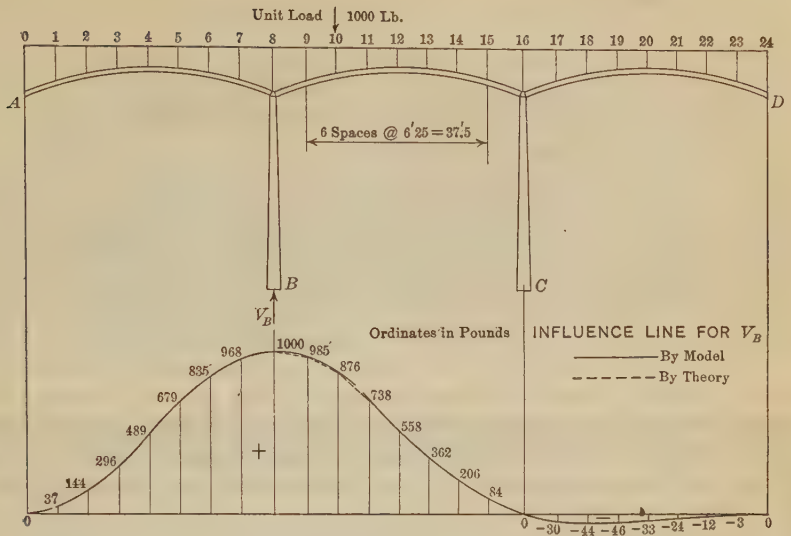

Unit Load 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
A
D
6 Spaces @ 6′25 = 37′5
B
C
V_B
Ordinates in Pounds
INFLUENCE LINE FOR V_B
By Model
By Theory
0 37 144 296 489 679 835 968 1000 985 876 738 558 362 206 84 0
0 −30 −44 −46 −33 −24 −12 −3 0

FIG. 42.

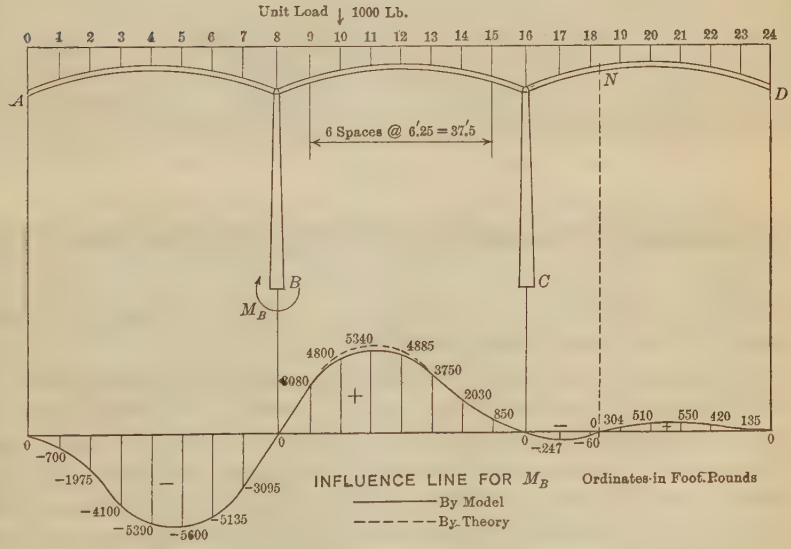

Unit Load 1000 Lb.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
A
N
D
6 Spaces @ 6′25 = 37′5
B
C
M_B
0 −700 −1975 −4100 −5390 −5600 −3095 0
0 2080 4800 5340 4885 3750 2030 850 0 −247 −60 0 304 510 550 420 135 0
INFLUENCE LINE FOR M_B
Ordinates in Foot-Pounds
By Model
By Theory

FIG. 43.

ported against any movement by single hinge pins placed on the lines of the reactions.*

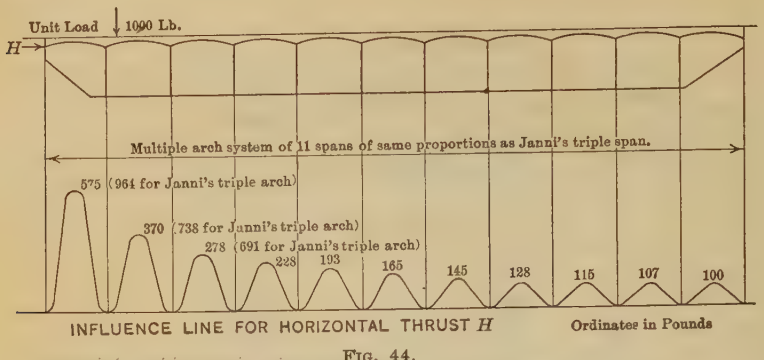


FIG. 44.

In Figs. 52, 53, 54, 55, 56, and 57, are shown photographs of structures in reinforced concrete, located with one exception (Fig. 53) in Europe. It is believed that when the American engineer becomes better acquainted with simpler methods for designing such indeterminate frames, he will be more ready to use them, particularly in cases where there is a demand for an artistic design of building frame.

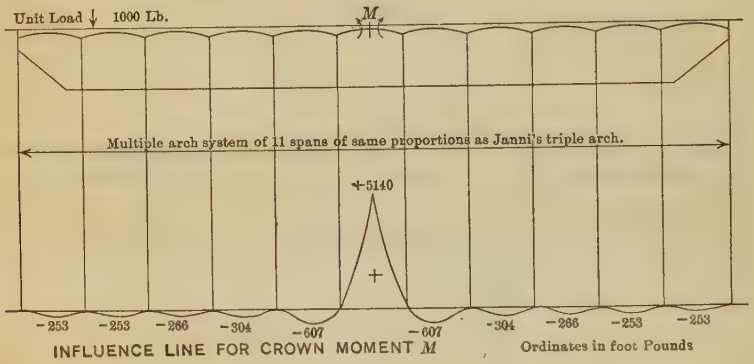


FIG. 45.

Following are observations regarding the method of solution by models as compared with other methods of calculation:

1.—The method of models is simple and has particular value because it facilitates the use of indeterminate structures, which class of structures are much more economical of material than simple determinate structures. Comparative studies of a number of statically determinate structures and indeterminate rigidly connected frames convince the speaker that frames which are rigidly connected member to member support their loads with minimum effort. The deflections under loads decrease as the rigidity of the structure

* Further applications of the methods of models for determining secondary stresses will be found in detail in *Proceedings*, Am. Concrete Inst., Vol. XVIII, pp. 58–82, and Vol. XIX, pp. 53–66.

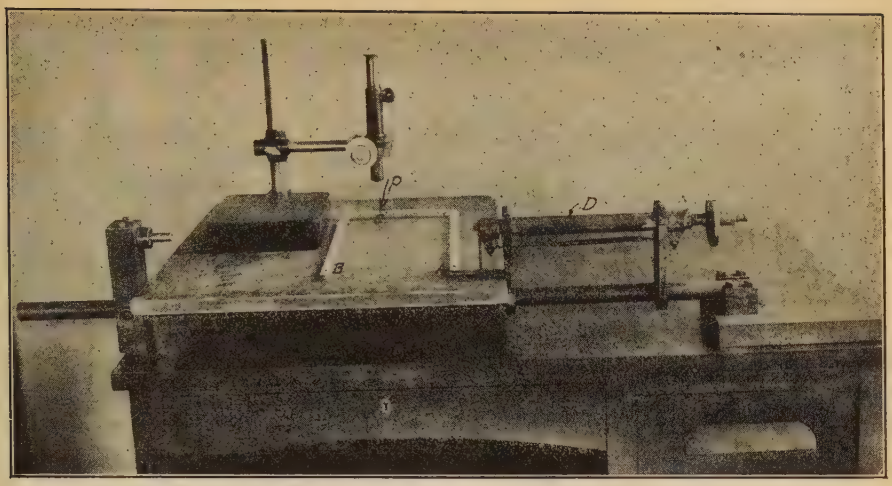


FIG. 46.—PROFESSOR RICHART'S APPARATUS FOR EXPERIMENTS WITH PORTALS.

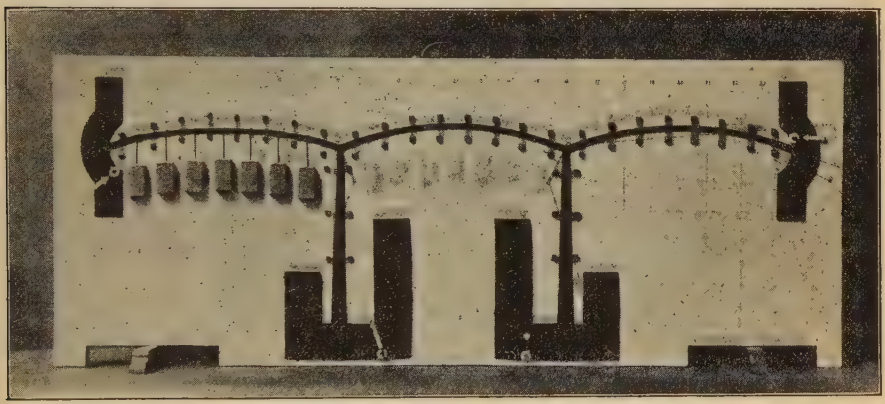


FIG. 47.—CELLULOID MODEL SHOWING ACTION OF JANNI ARCH WHEN LEFT-HAND SPAN IS LOADED.

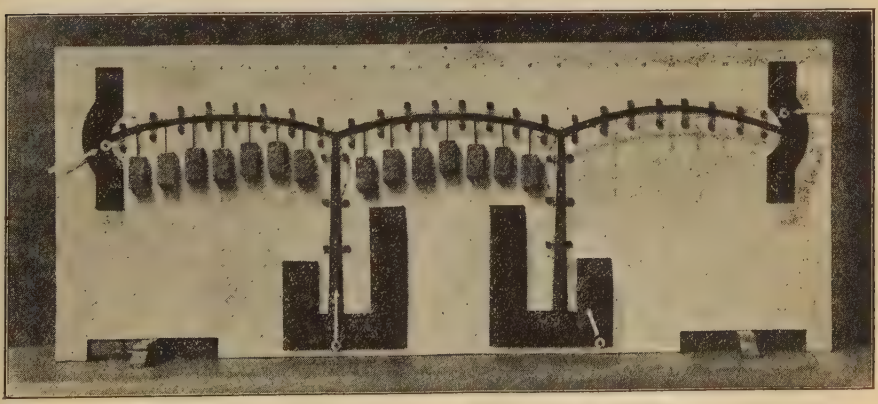


FIG. 48.—CELLULOID MODEL SHOWING ACTION OF JANNI ARCH WHEN TWO LEFT-HAND SPANS ARE LOADED.

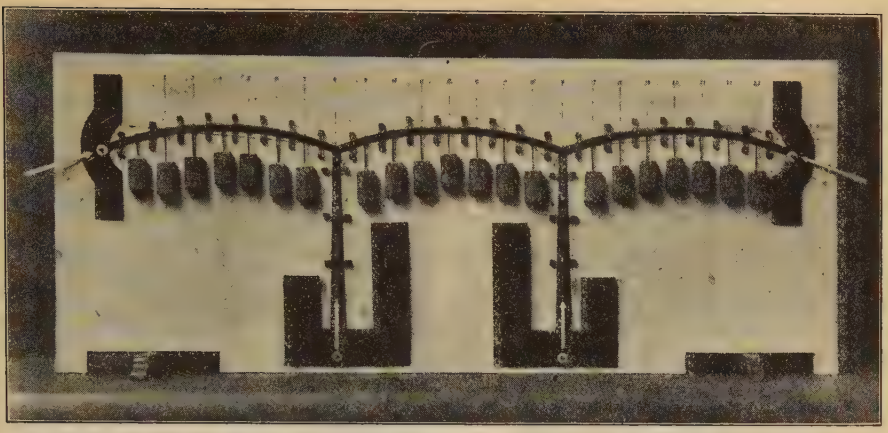


FIG. 49.—CELLULOID MODEL SHOWING ACTION OF JANNI ARCH WHEN ALL THREE SPANS ARE LOADED.

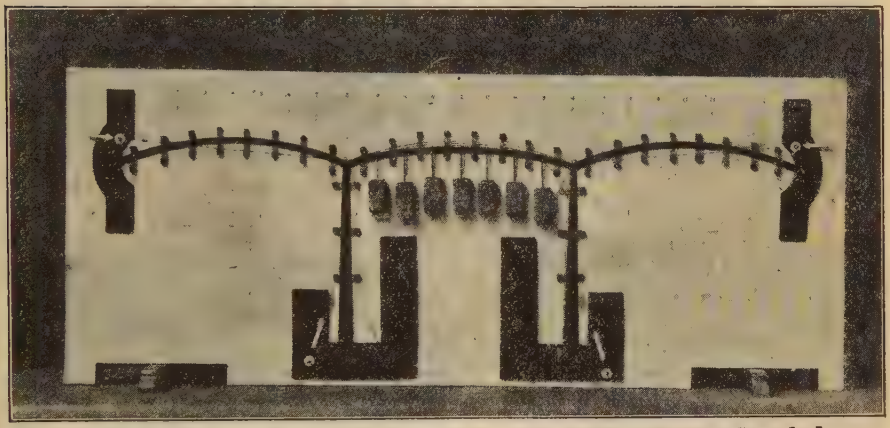


FIG. 50.—CELLULOID MODEL SHOWING ACTION OF JANNI ARCH WHEN MIDDLE SPAN IS LOADED.

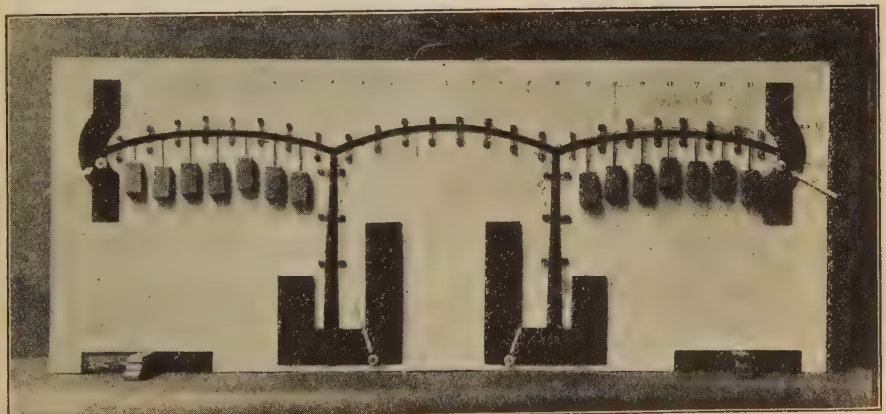


FIG. 51.—CELLULOID MODEL SHOWING ACTION OF JANNI ARCH WHEN TWO END SPANS ARE LOADED.

FIG. 52.—INTERIOR OF SHOP, SHOWING EUROPEAN CONCRETE PRACTICE.

FIG. 53.—CONCRETE BUILDING OF AMERICAN DESIGN.

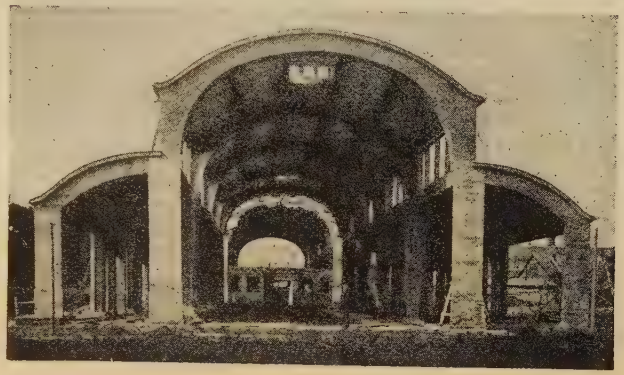


FIG. 54.—END VIEW, TRIPLE-SPAN EUROPEAN BUILDING.

FIG. 55.—CONCRETE ROOF, EUROPEAN PRACTICE.

FIG. 56.—MONOLITHIC ARCH AND SIDE COLUMN, EUROPEAN PRACTICE.

FIG. 57.—LARGE SPAN, CONCRETE BENTS, EUROPEAN PRACTICE.

increases; the external work done by the loads is so much the less; the corresponding internal elastic energy to be stored up by the stresses is equally reduced; whence a saving of material, and a reduction in the cost of the structure may follow. In a rigidly connected structure, not only is the work of supporting the load divided among all the members, but the total work to be done is less by reason of the rigidity.

2.—Observations of the deflections of models gives clear and true ideas of the action of elastic structures, and when these observations are made with instruments of precision, the results may be applied to designs immediately.

3.—The method, being closely related to Maxwell's theorem, serves as a means of readily checking many mathematical computations based on the theory of elasticity.

4.—The method is applicable to the determination of stresses in various types of structures such as conduits, sewer sections, continuous portals, arches of various types, subway cross-sections, and even secondary stresses in rigidly connected frames.

5.—In the usual methods of solving indeterminate structures, a greater number of simplifying assumptions are made than in the method using an elastic model.

6.—The method may be used in laboratory research to determine the manner in which stresses in members of structures are modified by gusset-plates, hinges, etc.

7.—For the calculation of indeterminate structures, the use of a model automatically gives the same results that mathematics yields only after the solution of many simultaneous linear equations. Conceivably, a machine might solve such equations, but no machine can be as perfect as the natural laws of deflection expressed in an elastic model of the structure itself. In the model, Nature both sets up the linear equations and solves them much more accurately than the algebraic equations can be solved with a special slide-rule.

8.—The models made of paper or celluloid follow the law of elasticity more closely than the structures of steel and concrete which they represent, and the laboratory errors are probably less than those in the assumptions on which the mathematical theory of structures is based.

9.—The use of models may reduce the tedious and disheartening calculations by complicated formulas, which have limited the freedom of engineers in creating economical and beautiful types of structures—types differing from the few common ones that have been developed largely by experience with full-sized construction.

10.—It is believed that the behavior of concrete arches corresponds to the elastic theory of structures as closely as the ratio of the deflections of a concrete arch corresponds to the ratio of the deflections of a perfectly elastic model. It is not considered essential that the engineer should be able to calculate the precise value of a single deflection of a point of a concrete arch,

but only that he estimate the proper ratio of the two deflections, d_1 and d_2, appearing in the basic equation of all methods, namely,

$$\text{Reaction component} = P\left(\frac{d_2}{d_1}\right)$$

C. W. HUDSON,* M. AM. SOC. C. E.—The paper by Mr. Janni and the discussion by Professor Beggs should be considered together, as they are very properly supplemental, each to each.

Mr. Janni's selection of the title for his paper is not descriptive. He gives very clearly a method for making the computations incident to a design of a series of three arches and the two intermediate supporting piers. Professor Beggs, by means of models as near as may be to scale, verifies these calculations by the relations of Hooke's law. The agreement in the determination of the statically indeterminate forces by Mr. Janni and Professor Beggs is remarkable when the difficulty of constructing truly representative models is contemplated.

The nature of the design assumed by the author to illustrate the manner of making the necessary computations is unusual. The arch sections are thin at the haunches and skewbacks and the intermediate piers high and slender. The intermediate piers are hardly of sufficient dimensions to act even as columns carrying only direct compression.

The author's attempt to draw general conclusions as to the forces and moments acting on a series of arches should not be given too much weight in their application to structures of more conventional dimensions. Professor Beggs showed by means of his remarkable models that the thrust and moment at the end of the end arches due to a load on one arch were considerable at all times, even for as large a series as eleven spans. The long slender piers, however, accentuate the amount of thrust transference from the loaded arch to the piers of the adjacent arches.

The necessity of keeping the total length of a series of arches within proper limits by means of intermediate abutment piers, is clearly brought out by Professor Beggs' model.

A. B. COHEN,† M. AM. SOC. C. E.—The author's effort to simplify the design of a reinforced concrete arch ring and to analyze a multiple system of arches, taking into account the continuity of action, is notable. The speaker has not compared the method of the analysis of the single arch that is founded on the theory of the ellipse of elasticity, with other methods in current use, but believes the consideration given in the paper to multiple arches will be a stimulus for further advance in such design.

During the past fifteen years a great improvement has been effected in the design of reinforced concrete arches. Seldom is an arch proposed which has a haunch thickness three or four times that at the crown. This unbalanced, uneconomical proportioning of former years gave low stresses at the haunch and excessive thrusts to piers and abutments; it is evident that the effective

* Prof. of Civ. Eng., Polytechnic Inst. of Brooklyn; Cons. Engr., New York, N. Y.
† Cons. Engr., New York, N. Y.

rise of an arch decreases as the haunch thickness increases without changing the curve of the intrados. A decided improvement obtains as a result of present efforts to fix the curve of the neutral axis to approximate the line of thrust caused by superimposed loading so that no tension exists in the arch ring and all sections are in more or less uniform compression. With the improvement of methods it is natural that certain immoderate liberties should be taken in designing. There have been misapplications of the material—an imitation in concrete of steel structures with diagonal members in tension, and other details which are peculiar to steel design, somewhat analogous to the misapplication of reinforced concrete in ship construction.

Some issue can be taken with Mr. Janni's analysis relating to the extraordinary slenderness of the pier, which slenderness would not be applicable to many multiple-arch bridge designs.

The speaker realizes that no attempt has been made to show the applicability of the analysis presented to the economic conditions governing construction, but believes these factors are so pertinent to the subject of multiple-arch design that attention should be called to them. The cost of labor and materials confines to a great extent the use of the arch to comparatively long spans. In the final analysis for economy there are factors in construction outweighing the design that results in minimum pier sections, for example, methods of construction would be required with the slender pier that would not only create unusual hazards, but require an excessive amount of arch centering.

The question of minimizing the hazards involves the design of the arches, the piers and their foundations, and the manner in which the work can best be subdivided. A series of arches and piers that require a large amount of centering to maintain their equilibrium until they become self-supporting cannot as a rule be considered economical. The design may impose so great a risk as to cause the experienced contractor to withdraw from competition. There are various ways to safeguard the work by constructing the arches in strips or ribs counterbalanced by the full weight of the pier. It is possible to construct a series of flat arches by the use of a small amount of steel arch centering without resorting to excessive pier sections. A low application of the horizontal thrust of the centering on an extended footing establishes equilibrium without the counterbalancing thrust of the centering of an adjacent arch in one stage of construction, and, in a second stage of greater moment, will give sufficient counterbalance to the thrust of a completed arch sector. This method practically eliminates all hazards in construction and gives a clear waterway for flood conditions.

The manner in which the work can be subdivided determines the size of the plant and the necessary amount of equipment and labor, which are the most important factors in determination of the cost. There is a definite relation between these factors and the magnitude of the work; for this reason it might be found more economical to subdivide the construction so as to adjust properly these factors even at the expense of adding materials. The well balanced design permits of continuous construction with a fixed amount of labor at all times.

Professor Beggs has demonstrated the invaluable use of celluloid models in the accurate analysis of arches and miscellaneous indeterminate structures. In showing the effect of a moving load on a series of arches a wide application of the use of models in reinforced concrete design is suggested particularly in the building of multiple arches to determine the stresses during the progress of the work. The contractor is interested in knowing just what hazards exist at all stages of his work. These hazards would properly affect his bidding and, therefore, the design should anticipate these conditions.

The assumption is made in the paper that the piers are elastic and extremely slender. The model demonstration has shown that the width of the pier footing required to hold the system of forces due to unbalanced loading on adjacent spans would limit the slenderness of the pier; but this width is governed by the overhang of a cantilever footing taking pressures of 3 tons per sq. ft., or more, which is itself quite limited.

T. D. Mylrea,* M. Am. Soc. C. E. (by letter).—The author's interesting application of the method of the ellipse of elasticity to the problem of the continuous arch on elastic piers is incomplete, unless the question of yielding foundations is adequately considered. The end span of a series with an elastically yielding foundation under its abutment loses rigidity at its ends, and suffers much as an intermediate span does from yielding supports. Yielding of the soil is much too important a matter to pass over with the scant treatment given in the paper.

The elastic weight of the foundation itself depends on a different constant from the modulus of elasticity of concrete, consequently, the value of G for the foundation must be modified accordingly. Therefore, after pointing out an error pertaining to rib-shortening which is becoming quite prevalent in this treatment of arches, and noting the method of including the effect of shear in a member, the writer proposes to discuss (1) a pier foundation; (2) the foundation of an abutment behind which the earth is deep enough to furnish practically equal resistance to horizontal movement from top to bottom of the abutment; (3) the foundation of an abutment behind which the earth is level and thus offers a variable resistance to horizontal movement; and (4) the necessary modifications in the elastic weights.

Rib-Shortening.—Under the heading, "Temperature", the author states: "The effect of temperature change, as well as that of shrinkage in concrete due both to the setting of the concrete and to the dead and live loads, is not presented in this paper". In finding the moment of inertia of the elastic weight of an element about a given line, the statical moment of the elastic weight about that line is considered as acting at the anti-pole of the line with respect to the ellipse of the element. This automatically takes into account rib-shortening due to the stresses produced by loads, dead or live; and the sentence quoted should read: "The effect of temperature change, as well as that of shrinkage in the concrete due to setting, is not presented in this paper." For proof of this, see the pamphlet by Dr. Postuvanschitz. The author himself shows (Paragraph 20), that the length of the minor axis of the ellipse of any

* Asst. Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

voussoir is determined by the shortening of that voussoir under axial stress.

The Effect of Shear.—In Paragraph 21, the length of the semi-major axis of the ellipse is given as:

$$\rho_1 = \sqrt{\frac{l^2}{12} + \frac{E}{S}\chi\rho^2}$$

the second term under the radical disappearing when shear is neglected. In the rare instance when the designer does not wish to neglect shear, a value for χ may be found, as follows:

χ is a coefficient expressing the relation between the true and the approximate shearing deformations; the true deformation is that which results from the actual distribution of unit shears across a section, and the approximate deformation is that which results from unit shears assumed to be the same at all points of a cross-section, and equal to the total shear divided by the area of the section. In Fig. 58, let,

Δy = transverse displacement of one end of the element with reference to the other;
ΔA = cross-sectional area of one fiber in the element;
v = unit shear on this fiber;
V = total shear on the section;
H = shearing modulus of elasticity;
Δs = length of the element under consideration.

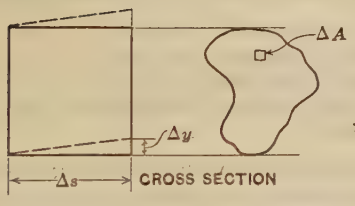


FIG. 58.

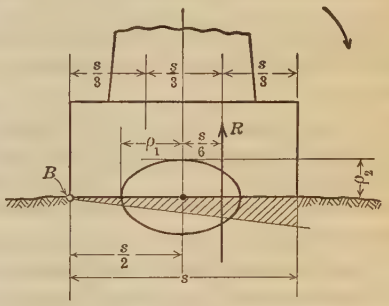


FIG. 59.

Due to the shear, one end of the elementary area, ΔA, is displaced vertically a distance, $\frac{v \cdot \Delta s}{H}$, with reference to the other end. The work of deformation on this fiber is, therefore,

$$\frac{v \cdot \Delta s}{H} \cdot v \cdot \Delta A = \frac{v^2 \cdot \Delta A \cdot \Delta s}{H}.*$$

Therefore:

$$V \cdot \Delta y = \sum \frac{v^2 \cdot \Delta A \cdot \Delta s}{H} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

* The factor, ½, always cancels in a discussion of virtual work, so it is omitted here.

or,

$$\Delta y = \frac{\Delta s}{V \cdot H} \sum v^2 \cdot \Delta A \quad \ldots\ldots (2)$$

which is the expression for the true shearing deformation.

If v is considered constant, Equation (2) becomes:

$$\Delta y = \frac{\Delta s}{V \cdot H} \cdot v^2 \cdot A \quad \ldots\ldots (3)$$

or, since,

$$v = \frac{V}{A}$$

$$\Delta y = \frac{V \cdot \Delta s}{A \cdot H} \quad \ldots\ldots (4)$$

which is the expression for the approximate shearing deformation. The relation between the values of Δy, as given in Equation (2) and Equation (4), is called χ. Dividing Equation (2) by Equation (4) gives,

$$\chi = \frac{A}{V^2} \cdot \Sigma v^2 \cdot \Delta A \quad \ldots\ldots (5)$$

This value, if substituted in the author's equation for ρ_1,* will take into account shearing distortions. As will be seen, the major axis of the ellipse would be somewhat increased, thus automatically including the shear distortion without further computation. Dr. Ritter states that, for rectangles, $\chi = \frac{6}{5}$, for circles and ellipses, $\chi = \frac{10}{9}$, and that for an I-section it is approximately equal to the total area of the section divided by the area of the web.

Elastic Foundations.—In discussing an arch resting on elastic supports, the author states:

"It is clear, therefore, that if in designing the arch, the ellipses of elasticity of the supports are assumed as if they were the ellipses of elasticity of two ideal additional voussoirs, * * * the resulting stresses will be for an arch under the influence of the elasticity of its supports" * * *.

The problem of an elastic foundation thus becomes one of finding the ellipse of an "ideal" element which has the same elastic properties as the foundation.

In Paragraph 25, Mr. Janni proves that: "It is, therefore, only when a force, F, acts along an axis of the ellipse that the displacement of A is parallel to the line of action of the force * * *." Therefore, if a force can be found which will produce a movement of the face on which it acts in the same direction as it acts, then the line of action of the force is along one of the axes of the ellipse. With one axis thus determined, the process can be repeated in a direction at right angles, thus locating the second axis. The center of the ellipse lies at their intersection.

Having found the center of the ellipse and the direction of the major and minor axes, the lengths of these axes may be found from a consideration of rotations; and the elastic weight is determined from the movement of the end of the element in the direction of an axial force. These two processes will be illustrated in the discussion of several types of foundations.

* See p. 1173.

Pier Footings.—Let Fig. 59 represent the footing of a pier, for which C equals the force, in pounds, necessary to depress, elastically, 1 sq. ft. of the soil a distance of 1 in.; D equals the elastic resistance to horizontal motion, in pounds per square foot of base per inch of movement*; and b equals the breadth of the footing.

By inspection it will be seen that to depress the footing vertically, a force will have to act vertically through the center of the base. One axis of the "soil ellipse" is, therefore, vertical and at the middle of the base. Also, to cause the footing to move horizontally, a force would have to act horizontally at the level of the bottom of the footing. The second axis is, therefore, horizontal and coincident with the bottom of the footing.

Now, imagine the footing to rotate in a clockwise direction about the point, B. The soil being elastic, a triangular pressure diagram is produced, and the resultant pressure, R, acts vertically through the outer edge of the middle-third of the base. It is proved by the author† that "a terminal point, A, of a plane elastic system acted on by a force, R, rigidly connected with it, will rotate about the anti-pole (with respect to the ellipse of elasticity of the system) of the line of action of R." B is, therefore, the anti-pole of the force, R, with respect to the "soil ellipse".

It would serve the same purpose to assume the line of action of a force, and then determine its center of rotation,‡ but in the case at hand it is simpler to assume the center of rotation and determine the corresponding force. Any point might be chosen, but that one is best which makes the calculation of the corresponding force easiest. For any small rotation about the point, B, the resisting force of the earth is wholly normal to the base, and the resultant is easily found.

From a consideration of the author's Fig. 9 and Fig. 59:

$$\frac{s}{6} : \rho_1 = \rho_1 : \frac{s}{2} \quad \ldots\ldots (6)$$

from which,

$$\rho_1 = s\sqrt{\frac{1}{12}} \quad \ldots\ldots (7)$$

Thus, the length of one of the axes is determined.

The elastic weight represented by the "soil ellipse" is determined as follows.

$$\text{Depression under vertical force, } F, = \frac{F}{C\,b\,s} \quad \ldots\ldots (8)$$

In Paragraph 20, the author deduces an expression for the movement of the end of an element in the direction of an axial force, which for the present case may be written:

$$\text{Depression under a vertical force, } F, = F \,.\, G \,.\, \rho_1{}^2 \quad \ldots\ldots (9)$$

* An elastic movement implies that the upper surface of an earth element below the base moves horizontally with respect to the lower surface of such an element. Obviously, the maximum value of D would be equal to the force which would produce sliding, subsequent movement being no longer elastic.

† See p. 1171.

‡ See "The Resistance of a Group of Piles", by H. M. Westergaard, Assoc. M. Am. Soc. C. E., *Journal*, Western Society of Engineers, December, 1917, Article 4, p. 707.

Combining Equation (8) and Equation (9):

$$F \cdot G \cdot \rho_1^2 = \frac{F}{C\,b\,s} \quad \ldots\ldots (10)$$

The value of ρ_1 is known from Equation (7), therefore, the elastic weight is:

$$G = \frac{12}{C\,b\,s^3} \quad \ldots\ldots (11)$$

The semi-axis, ρ_2, may be computed just as ρ_1 was. Assume a horizontal force, F, acting on the footing at the ground level:

$$\text{Horizontal movement due to force, } F = \frac{F}{D\,b\,s} \quad \ldots\ldots (12)$$

but, as before,

$$\text{Horizontal movement due to force, } F = F \cdot G \cdot \rho_2^2 \quad \ldots\ldots (13)$$

Combining Equation (12) and Equation (13):

$$F \cdot G \cdot \rho_2^2 = \frac{F}{D\,b\,s} \quad \ldots\ldots (14)$$

from which,

$$\rho_2^2 = \frac{F}{F \cdot G \cdot D\,b\,s} \quad \ldots\ldots (15)$$

But, if $G = \frac{12}{C\,b\,s^3}$ (see Equation (11)):

$$\rho_2^2 = \frac{C\,b\,s^3}{12\,D\,b\,s} = \frac{C}{D} \cdot \frac{s^2}{12} = \frac{C}{D}\,\rho_1^2 \quad \ldots\ldots (16)$$

Therefore,

$$\rho_2 = \rho_1 \sqrt{\frac{C}{D}} \quad \ldots\ldots (17)$$

Thus, Equations (7), (11), and (17) together completely determine the "ideal" element which has the same elastic properties as this foundation.

Before proceeding with further examples, two interesting facts should be noticed:

1.—The author shows on page 1172 that:

$$\rho = \frac{I}{A} \quad \ldots\ldots (18)$$

For a homogeneous rectangular section, this would become:

$$\rho = \frac{b\,d^3}{12} \div b\,d = \frac{d^2}{12} \quad \ldots\ldots (19)$$

from which,

$$\rho = d \sqrt{\frac{1}{12}} \quad \ldots\ldots (20)$$

This furnishes an interesting comparison with the value of ρ_1 which Mr. Janni deduces on page 1173, where he shows that:

$$\rho_1 = l \sqrt{\frac{1}{12}} \quad \ldots\ldots (21)$$

In other words, for homogeneous material and rectangular cross-sections, the axes of the ellipse are proportional to d and l, respectively.

2.—If in the value of $G = \frac{d\,s}{E\,I}$, which the author uses, the value, $I = \frac{b\,d^3}{12}$ is substituted:

$$G = \frac{12}{E\,b\,d^3} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Comparing this with Equation (11) shows why the constant, C, may be termed the "modulus of elasticity of the soil in compression". This was stated, without proof, in the author's previous treatment of the ellipse.* Similarly, D might be called the "modulus of elasticity of the soil" in shear.

A critical examination of Equation (17) leads to the conclusions that the smaller the resistance, D, to horizontal moment, the greater the vertical semi-axis, ρ_2. This may seem anomalous, but it must be remembered that the value of G was computed from C, and the result used, in conjunction with D, to determine ρ_2. Now, C and D are "moduli of elasticity" which may differ greatly. Therefore, either the value of G, computed from C, must be multiplied by $\frac{C}{D}$ when dealing with D, or else the length of the vertical axis of the ellipse must be changed to avoid changing G. This explains the apparent anomaly. Thus, if D should be zero, the vertical axis would be infinitely long, and the anti-poles of all lines of action with reference to the soil ellipse would lie on this axis. All forces other than truly vertical ones would cause the base to skid. On the contrary, if horizontal movements are so small as to be negligible, it implies an infinite value of D; and from Equation (17) it will be seen that in this case the length of the vertical axis would be zero. This is also stated, without proof, by Hool, in the section of his book referred to previously. If C and D are equal, the ellipse becomes a circle.

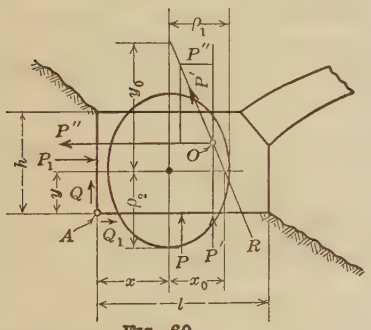


FIG. 60.

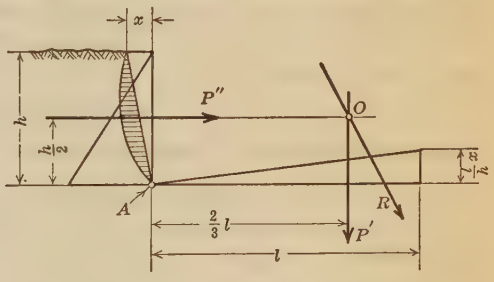


FIG. 61.

Buried Abutments.—In Fig. 60, let it again be assumed that C and D are the moduli of elasticity of the soil in compression and shear, respectively. Let it also be assumed that the slope behind the abutment is undisturbed soil such that C may be considered constant at all points.

* Hool's "Reinforced Concrete Construction", Vol. III, Article 48, p. 153.

If the abutment were pushed vertically downward a distance, e, the motion would be resisted by the normal force, $P = C\,b\,l\,e$, on the bottom, and by the vertical force, $Q = d\,b\,h\,e$, on the back. The vertical axis of the ellipse would lie along the resultant of the vertical forces, P and Q, at a distance, x, from the back. If the abutment were pushed horizontally a distance, f, the motion would be resisted by the normal force, $P_1 = C\,b\,h\,f$, on the back, and by the horizontal force, $Q_1 = D\,b\,l\,f$, on the base. The horizontal axis of the ellipse would lie along the resultant of P_1 and Q_1, at a distance, y, from the base, and the center of the ellipse would be at the intersection of the axes.

If the abutment be assumed to rotate in a clockwise direction about the point, A, a triangular pressure diagram under the base results; and the resultant vertical pressure, P', acts at the edge of the middle-third of the base. There is also a resultant tension (release of compression), P'', normal to the back of the abutment, which, due to its triangular stress diagram, acts at the upper edge of the middle-third of the back. Then:

$$P' : P'' = l^2 : h^2 \qquad (23)$$

The resultant of P' and P'' acts through the point, O, and as its direction may be determined from Equation (23), the actual values of P' and P'' need not be computed.

As in the discussion of the pier, the point, A, is the anti-pole of the force, R, with respect to the soil ellipse, and the lengths of the semi-major and minor axes may be found from the equations:

$$x = \frac{\rho_1^2}{x_0}, \text{ or } \rho_1 = \sqrt{x\,x_0} \qquad (24)$$

$$y = \frac{\rho_2^2}{y_0}, \text{ or } \rho_2 = \sqrt{y\,y_0} \qquad (25)$$

These equations are only adaptations of Equation (6).

To find the elastic weight, assume the abutment to be pushed vertically downward a distance, e. Then, as in Equation (8),

$$e = \frac{P + Q}{(C\,l + D\,h)\,b} \qquad (26)$$

but, as in Equation (9),

$$e = (P + Q)\,G\,\rho_1^2 \qquad (27)$$

Equating Equations (26) and (27),

$$(P + Q)\,G\,\rho_1^2 = \frac{P + Q}{(C\,l + D\,h)\,b} \qquad (28)$$

from which,

$$G = \frac{1}{(C\,l + D\,h)\,b\,\rho_1^2} \qquad (29)$$

In such an abutment as this, it is permissible to consider the resistance, D, as equal to zero, and the few necessary changes in the equations can easily be made by the designer. The results are on the side of safety. In the case of the pier discussed previously, it is not permissible thus to neglect the resistance, D, unless the designer is sure that the pier will never be subject to any but vertical

axial forces. The practice of building continuous arches on piers supported only on vertical piles is certainly open to question.

If desired, the elasticity of the abutment itself can be included. The axes of this abutment ellipse will be vertical and horizontal; the lengths of the semi-axes will be $h\sqrt{\frac{1}{12}}$ and $l\sqrt{\frac{1}{12}}$, respectively (see Equations (20) and (21)), and the elastic weight will be,

$$G = \frac{12}{E_c \cdot l \cdot h \cdot b} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

Abutments the Tops of Which Are Level with the Ground.—If the material behind the abutment is level with its top, it is certain that C is not constant from top to bottom of the back of the abutment, as the upper layer of earth cannot resist a horizontal force. The extreme case in actual practice would be the one in which C and D both varied from zero at the top to a maximum at the bottom. All other cases would probably be intermediate between this and the one in which C and D were constant. Therefore, let it be assumed in the following discussion that C and D vary directly from zero at the top to a maximum at the bottom.

Under the influence of an assumed vertical movement of the abutment the force, Q, is only one-half as great as in the previous example. Therefore, the vertical axis of the ellipse passes more nearly through the center of the base. Under an assumed horizontal movement, the force, P_1, is likewise only one-half as great as previously, and it acts at $h\sqrt{3}$ above the point, A, instead of half way up. Hence, the center of the ellipse is lower and closer to a vertical line through the middle of the base. Its exact position will be at the intersection of the resultants of the vertical and horizontal forces, as in the "buried" abutment.

In this case, for convenience, assume a counterclockwise rotation about the point, A, sufficient to move the top of the abutment a distance, x, toward the left (Fig. 61). The forces, P, Q, P_1, and Q_1, have been omitted, for the sake of clearness.

$$P' = \frac{1}{2}\, b\, C\, \frac{l}{h}\, x \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

It acts at a distance $\frac{2}{3}\, l$ from the point, A.

Due to this same rotation, compression is developed, acting against the back of the abutment. At a distance, y, above the point, A, the horizontal movement is $\frac{y}{h}\, x$. Owing to the assumption that the value of C varies from zero at the top to C at the bottom, the value of C_y at any point, y, is,

$$C_y = C\left(1 - \frac{y}{h}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

Therefore, the unit pressure, p, at y is,

$$p = \frac{y}{h} \cdot x \cdot C\left(1 - \frac{y}{h}\right) = C \cdot x \cdot \left\{\frac{y}{h} - \left(\frac{y}{h}\right)^2\right\} \quad \ldots\ldots\ldots\ldots (33)$$

This is the equation of a parabola with a horizontal axis at a distance, $\frac{h}{2}$, above the point, A (Fig. 61). The total pressure, P'', acting toward the right, will be:

$$P'' = C b \cdot x \cdot \int_0^h \left\{ \frac{y}{h} - \left(\frac{y}{h}\right)^2 \right\} dy \quad \ldots\ldots(34)$$

from which,

$$P'' = \frac{1}{6} C \cdot b \cdot x \cdot h \quad \ldots\ldots(35)$$

The resultant of P' and P'' will act as shown in Fig. 61, and the lengths of the semi-axes of the foundation ellipse may be determined from the intercepts of R on these axes, as shown in Equations (24) and (25).

The elastic weight of this foundation may be found by assuming the abutment to be pushed downward a distance, e, from which (see Equation 26):

$$e = \frac{P + Q}{(C l + \frac{1}{2} D h)\, b} \quad \ldots\ldots(36)$$

but,

$$e = (P + Q)\, G\, \rho_1^2 \quad \ldots\ldots(37)$$

Combining Equations (36) and (37):

$$G = \frac{1}{(C l + \frac{1}{2} d h)\, b\, \rho_1^2} \quad \ldots\ldots(38)$$

The Elastic Weights.—For a homogeneous member the expression for the elastic weight is:

$$G = \frac{d s}{E I} \quad \ldots\ldots(39)$$

If the member is not homogeneous, a "transformed section", which is homogeneous, may be substituted. The author, for example, substitutes a transformed section of concrete for his combination steel and concrete member. He had then to consider only the modulus of elasticity of concrete. An inspection of Equation (39) will show that the elastic weight of an elementary length of a homogeneous steel member will be only $\frac{E_c}{E_s}$, or approximately one-fifteenth of that of the same elementary length of a concrete member of the same cross-section. Therefore, if some members of a framework are composed of concrete (or reinforced concrete "transformed") and others are of steel, either the elastic weights of the steel members will have to be multiplied by 15 or the elastic weights of the concrete members divided by 15, in order to have them all in the same terms. This is the significance of the previous statement, that "either the value of G, computed from C, must be multiplied by $\frac{C}{D}$ when dealing with D", etc.

In a similar manner, the value of G in these pier and abutment examples has been deduced from a "modulus of elasticity", C. Therefore, to use the

"soil ellipses" as found when combining them with the ellipses of a concrete arch, the value of G in Equations (11), (29), and (38), must first be multiplied by $\frac{C}{E_c}$. The ellipses will remain as found.

Conclusion.—It should be pointed out, however, that values of C can only be determined approximately. Whether or not tests on small areas are reliable guides for computing depressions of large areas has not thus far been settled. Moreover, the value of D is very difficult to obtain. Therefore, in the charm of analytical intricacy, the engineer must not forget that some of his constants may be only approximations.

CHARLES S. WHITNEY,* M. AM. SOC. C. E. (by letter).—This paper is an excellent and interesting presentation of the Culmann-Ritter method of the ellipse of elasticity. This method has also been presented recently by C. B. McCullough, Assoc. M. Am. Soc. C. E.,† M. Paul Sejourne,‡ and M. M. Gay.§ The author's treatment of the design of multiple-arch systems leaves much to be desired and the writer would take exception to his conclusions, which are not at all supported by the facts demonstrated in the paper.

The effect of loads on the center span only has been considered. For a complete analysis from which the three arch spans and the piers could be properly designed, it would be necessary to determine the effect, on both the center and end spans, of loads on all spans. For this complete solution, there are other methods which require less labor than the purely graphical one presented.

In order to show the importance of the effect on both the center and end spans of loads on adjacent spans, the writer has given in Fig. 63 influence lines for the three-span system shown in Fig. 62 which has been analyzed by

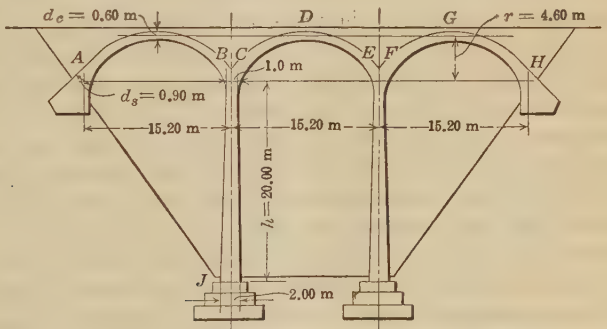


FIG. 62.—DIMENSIONS OF ARCHES AND PIERS.

Ernst Pichl.‖ The ribs and piers have a uniform width of 4 m. It will be noted that high positive moments at the springing line and negative moments at the crown are produced by the loads on adjacent spans. These moments are

* Cons. Engr., Milwaukee, Wis.
† "Reinforced Concrete and Masonry Structures", Hool and Kinne, p. 497.
‡ "Grandes Voutes", Vol VI.
§ "Ponts en Maçonnerie."
‖ "Der durchgehende gelenklose Bogen auf elastischen Stützen."

due largely to the horizontal thrust which is transmitted from span to span on account of the elasticity of the piers. The rotation of the head of the piers due to bending of the piers has also an effect on the bending moment in the

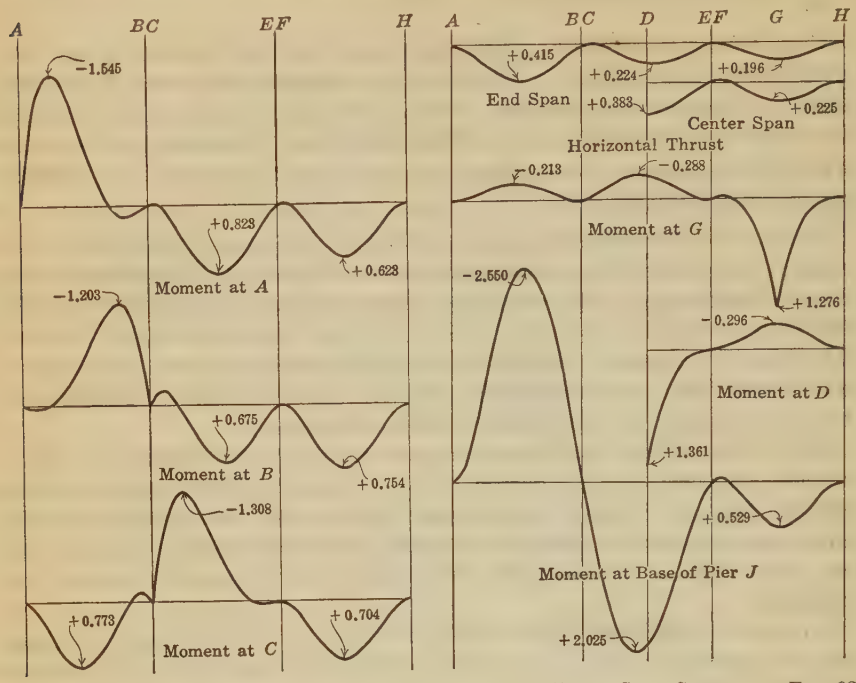


FIG. 63.—INFLUENCE LINES FOR MOMENTS AND THRUSTS, THREE-SPAN SYSTEM OF FIG. 62.

spans, and this effect decreases as the piers become more rigid. An interesting study of this effect is made by Pichl. After analyzing the structure shown in Fig. 62, he increased the dimensions of the piers so that they were 2.5 m. thick by 4 m. wide at the top and 5 m. thick by 7 m. wide at the base. The height of the piers and the dimensions of the ribs were kept the same. With these new proportions, he found that the effect of the rotation of the pier tops could be neglected without appreciable error and that it was necessary, therefore, to consider only the horizontal displacement of the pier tops.

The accuracy of any approximate method of analyzing multiple-arch systems depends entirely on the relative proportions of the arches and their supports or on their relative rigidity. If Mr. Janni wished to demonstrate the accuracy of the simplification which he attributes to Professor Guidi, he was unfortunate in his selection of an example. With the very flat arches and extremely slender piers of the system he has analyzed, the addition of similar spans at the end would materially reduce the horizontal thrust in the loaded span and would increase the bending in the ribs. With the spans and piers as shown, it would not be sufficient to consider only three adjacent arches. It might be sufficient to consider the group of three spans only if the outer ends were considered as fixed in direction alone, taking account of the horizontal

displacements due to the flexibility of the spans beyond. The effect of the rotation of the top of the second pier from the span under consideration would probably be negligible, so that for the center span of the three, the results would be accurate. In practice, such slender proportions would not, ordinarily, even be considered; still, an accurate, complete method of analysis is desirable if for no other reason than to determine the limits of accuracy of the approximate methods.

There are a number of approximate methods which are probably accurate enough for practical cases, but there is little information as to the limits within which they may be used. The particular need is for such information. Using the arch ribs as established in his paper "Design of Symmetrical Concrete Arches",* the writer has developed a new, simple, and accurate method of analyzing two symmetrical ribs with an elastic pier between. He has also applied it to the three-span system and believes it to be simpler than any other method he has seen.

In designing hingeless arch systems, the possibility of a rotation of the abutments and pier bases should always be considered. It may safely be assumed that they will rotate to some extent, as there is no such thing as a perfectly rigid support. Every effort should be made to design the abutments and bases so that the dead load pressures will be distributed uniformly in order that the permanent load will cause no rotation. When tall piers are used, a careful study should be made to determine whether or not the bases are really fixed as assumed. If it is necessary to raise the springing line high above the pier bases, the unbalanced horizontal thrust from live load on one span may produce such a tremendous moment that the pier base will rock on the foundation. The stiffer the shaft of the pier, the greater will be the moment at its base. Any shifting of resultant pressure will cause a rotation due to yielding foundations. The effect of this rotation is more serious with a tall pier than with an abutment because, in the former case, it produces also a horizontal displacement of the end of the rib which is exaggerated by the height of the pier.

On account of the large number of long multiple-span arch bridges which are being built and which will be built in the future, there is a definite need for complete, dependable data on the effect of movements of piers due to the elasticity of the supporting foundation or of the piers. These data must be secured by measurements in the field and by a comprehensive theoretical investigation to determine what methods of analysis may properly be used for different cases. The writer is of the opinion that such an investigation will show that reasonably simple methods of design may be used for any arch systems which are likely to be built.

Dr. Ing. W. Schachenmeier,† (by letter).—This interesting paper has induced the writer to offer some discussion that is to be taken only as a stimulation for further investigation.

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* See p. 931.

† Prof., Polytechnical School, Munich, Germany.

The writer fully agrees with the author's conclusions, especially that it is permissible to consider only three adjacent arches of the whole multiple system, assuming their outer ends to be fixed. The elegance of the graphical method applied by the author, and the clearness of his description also deserve appreciation. According to the writer's experience, however, the use of the graphical method, based essentially on the drawing of funicular polygons, does not guarantee the necessary exactness in the case of longer spans, that is, in those very cases in which the application of the theory of elasticity is not only permissible, but necessary. The designer will experience great disappointment and the results may be quite inconsistent with the general rules of equilibrium of arches, if he relies on the drawing of funicular polygons, no matter whether it concerns a simple arch or a multiple-arch system. Therefore, notwithstanding the clearness of the graphical method, it must be combined with the analytical method in all important cases. Whether the latter method becomes cumbersome and more laborious than the graphical method, depends largely on the training of the designer.

In addition, the analytical method will afford an insight into the nature of the problem that the graphical method never can give. The analytical method brings out the relative influence of the different causes, that is, the elasticity of the arch, of the pier, and of the soil. In some cases, especially for very long spans or flat arches, it may even be important to take into account the influence of the change of span and rise on the stresses. This is quite feasible with the analytical, but not so, the writer believes, with the graphical method.

Those who have to deal with the multiple-arch system and desire to comprehend its characteristics, should study the excellent paper of Fr. Engesser entitled, "Ueber Bogenbrücken mit elastischen Pfeilern (Bogenreihen)".* Twenty-three years ago, Engesser, a most prominent scientist and civil engineer, made an exhaustive investigation of the multiple-arch system in comparison with the single arch. His treatise not only concerns arches of concrete and masonry, but of every other possible material and treats of arches with three hinges, two hinges, and no hinges. His methods, which are entirely analytical, give a clear insight into the static and elastic properties of the system. The results are condensed in formulas of comparative simplicity and of a form which permits of discerning the various influences, both qualitatively and quantitatively.

Engesser's results are probably unknown to most American engineers and, in the writer's opinion, their translation into the English language would be well justified.

The writer desires to emphasize that generally speaking he does not disparage the graphical method, but believes in a judicious combination of the two methods. He appreciates greatly the scientific value of the author's treatise.

* *Zeitschrift für Bauwesen*, 1901, p. 311.

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THE MODERN CITY AND THE ENGINEER'S RELATION TO IT

ADDRESS AT THE ANNUAL CONVENTION AT CINCINNATI, OHIO, APRIL 22, 1925

By Robert Ridgway,* President, Am. Soc. C. E.

Perhaps the most notable effect of the application of those laws of Nature which have been brought to light by the patient investigations of the scientist during the past century and a half is evidenced in the wonderful growth of cities everywhere, and, as the engineer has contributed so largely to this result, I have chosen to make the theme of my address: "The Modern City and the Engineer's Relation to It," realizing that the thoughts expressed are not original with me, but have long been in the minds of others.

In the simple days when our Federal Constitution was adopted, we were essentially an agricultural people and manufacturing was but an incident. The planter was the man of affairs and when he came to town he was shown the deference befitting his standing in the community. Then, the steam engine began to play its important part in the affairs of the nation and among other applications made transportation over long distances easier. This was followed by the "magnetic telegraph", the result of the good work of Henry and Morse, and, since then, one invention has followed another at an increasing rate up to this time. We do not often stop to think how recent and how new nearly all the inventions which contribute so much to our comfort and well-being really are. It is necessary to go back only to the infancy of those now living to realize this. When I was born, the telegraph was still regarded as a marvel and the locomotive was rather a primitive machine if compared with the great engines of to-day. No trans-Atlantic cable had been successfully laid. The telephone was unthought of and electric lighting and traction were

* Chf. Engr., Board of Transportation, City of New York, New York, N. Y.

still the dreams of scientists. Water powers were largely going to waste because, although their potential value was understood, the market for the power did not exist and methods of harnessing it for the production of hydro-electric energy were unknown.

The average man, then much more than now, was reluctant to accept these new ideas, which disturbed his habit of thought and action. We are amused when we read of the opposition, violent at times, to the introduction of inventions that later contributed so much to the upbuilding of communities and are now regarded as commonplace. The following extract from "The World To-morrow" is of interest as illuminating this state of mind:

"* * * the following resolution passed in 1828 by the School Board of Lancaster, Ohio, and once quoted by Dr. Fosdick: 'You are welcome to use the school-house to debate all proper questions in. But such things as railroads and telegraphs are impossible, and rank infidelity. There is nothing in the word of God about them. If God had designed that his intelligent creatures should travel at the frightful speed of fifteen miles an hour by steam, He would have foretold it by the mouth of His holy prophets. It is a device of Satan to carry the souls of the faithful down to Hell.'"

Had one possessed a modern radio set in those days and had dared to exhibit it in action, I fear he would have taken his life in his hands. When one of my distinguished predecessors, after his retirement from the office of President, took up in a serious way the problem of æronautics, there were those, engineers as well as others, who showed concern for him and wondered whether he had taken leave of his senses. Most people then were content to wait until they had departed from this world before learning to fly. We are amused at these evidences of conservatism, but we must remember that the mass of mankind is naturally conservative by nature, thus keeping the world within bounds; otherwise, it would be taking up every ill-digested idea that was plausibly presented to it. Most men have neither the time nor the qualifications to rightly analyze every new condition as it develops and to differentiate between the few ideas that are good and the many that are worthless. How many fortunes have been lost by investors in patents which were intended by their inventors and claimed by their promoters to revolutionize established methods and which failed to accomplish their purposes because of basic defects?

Modern man is inclined to take as a matter of course all the many developments of scientific research as soon as they become available, and they come at an ever-increasing rate. It is only the thinking man who marvels at the wonderful results which are the outcome of patient study by the scientist and faithful application by the engineer. It is singular how soon men adapt themselves to the changes brought about by new inventions. Most men accept them as they accept sunshine and rain without attempting to understand them. To the average man even the radio has ceased to be a nine-days wonder and an aeroplane barely receives passing notice from him. He knows nothing of the pioneer work of Hertz and has never heard of the contributions to the art of flying made by Langley and Chanute under discouraging conditions before the Wrights made flying an accomplished fact. The pure scientist has been likened to a dreamer interested in a subject or investigation for itself alone without any definite idea of its practical application. He dis-

covers the fact and thus enables the engineer to apply it to a definite and useful purpose. It is extremely rare to find the pure scientist and the applied scientist or engineer combined in one personality. Times without number the former has not received from the world that recognition which was his due. His work in the quiet of the study or the laboratory is not, as a rule, spectacular, and it must be remembered that few scientific truths are discovered by chance.

"Patient research, toil and the effort of years constitute the denominator in the fraction which represents the formula of scientific accomplishment and in this fraction the numerator looms large with dreams of hope, of success, and of future achievement. Without hope, without vision, and without dreams success would ever be beyond attainment."

To myself I have often pictured the world as it might now be if scientific development had begun earlier and had progressed at a less feverish rate than during the past few generations. When, with members of the Board of Direction of the Society, I visited Muscle Shoals on the Tennessee River in 1923, I could not help thinking of what a different slant our history might have taken if the aboriginal red man had been gifted with an understanding of what the energy represented by that falling water meant and had been able to apply that knowledge for the good of his race.

We should not give those of the past century the entire credit for all the wonderful developments which have been made, because others were patiently laying the foundations for them centuries ago and without the good work which they did we would be far behind the point that has now been attained. The first steps in progress tend ever toward extravagance. The marvellous development of our own country during the Nineteenth Century was made at great expense. Our natural resources, such as timber, coal, and wild animal life, appeared to be inexhaustible and shocking waste resulted. Now that these resources have been depleted, and the exhaustion of some of them is in sight, a policy of conservation becomes the order of the day. Europe long since learned the lesson of economy, but we are fast learning the same lesson which was forced on the older countries long before it was realized or heeded here. The world looks to the scientist, to the engineer, and to the chemist for assistance in learning how to make the best use of our remaining natural wealth. Their problem, however, goes further. They must point the way to the utilization of substances and materials which are now considered worthless. Their's is the task of the alchemist.

Because of the rapid rate at which improvements are being made and because the application of one invention always seems to lead to several others, one wonders what is in store for the next generation. What a treat awaits those who are now students in college and what opportunities for usefulness they will have as a result of the research work which is now going on!

Most of the inventions of the future will probably not be new and basic, but will come as an evolution of those previously made. Many will operate toward the elimination of wasteful methods, making fuel, for instance, do a greater amount of work per unit. Some will enter a new field and, as an illustration, there may be cited the case of the recent development of the Flettner rotor

ship which is propelled by the wind, without sails, utilizing the tendency of a rotating cylinder to create a vacuum on one side of the cylinders. It has been suggested that this principle might be applied to the development of a type of helicopter, with the idea that if the experiment were successful it would have a tendency to revolutionize the whole theory of heavier-than-air flying machines.

The habits and customs of our people as well as their social structure have been profoundly influenced by our wonderful material development. We have been changed from an essentially agricultural people to an industrial nation. When the first Federal Census was taken in 1790, probably more than 90% of the total population of about 4 000 000 was rural and less than 10% was urban, that is, dwellers in cities of 2 500 or more. Philadelphia, with a population of 42 520, was the largest city and New York was second with 33 131 inhabitants. In 1920, only 46% of the 113 000 000 inhabitants of Continental United States was rural and 54% was living in cities or towns of 2 500 or more inhabitants. Like the advance in applied science, the change from the rural to the urban condition has been going on at an ever-increasing rate, but some of those who have analyzed the figures believe there are signs of a slowing down of this rate. The automobile, with improved highways, the telephone, and the radio, together with labor-saving farm machinery, with improved living conditions on the farm, have come to make country life more comfortable to such an extent that the migration to the cities is being gradually checked.

These changed conditions are not peculiar to our own country and Canada. Europe and the world over show the same tendency, thus indicating that the fundamental causes are not due alone to the development of a relatively new country, but to the growing substitution of machinery for hand work and the creation of new conditions of living. The change, however, is probably more pronounced in newer countries like our own.

A few examples of the recent growth of the leading cities are given in Table 1.

TABLE 1.

City.	1880.	1890.	1900.	1910.	1920.
New York, N. Y.	1 911 698	2 507 414	3 437 202	4 766 883	5 620 048
Chicago, Ill.	503 185	1 099 850	1 698 575	2 185 283	2 701 705
Philadelphia, Pa.	847 170	1 046 964	1 293 697	1 549 008	1 823 779
Boston, Mass.	362 839	448 477	560 893	670 585	748 060
Baltimore, Md.	332 313	434 439	508 957	558 485	733 826
Los Angeles, Calif.	11 183	50 395	102 479	319 198	576 673
New Orleans, La.	216 090	242 039	287 104	339 075	387 219
Seattle, Wash.	3 533	42 837	80 671	237 194	315 312

The growth of the cities, however, does not tell the whole story. In the old days, the city was little more than a layout of streets and a collection of houses. Business offices and shops were under the same roof or within walking distance of the homes of both employer and employee. The homes and shops were

lighted by candles or lamps. The water supply, in most cases far from wholesome, came from wells or town pumps, and cesspools did in part the work of the modern sewer. Where paving existed at all, it was rough and generally in poor repair. Live stock roamed the streets which were cleaned only occasionally. Living, when viewed from the standpoint of to-day, was primitive. The relations between private and public health were unknown, and the police power of the body politic was confined to the simple preservation of the peace.

The modern city is a most complicated organism. Its streets and buildings are but an expression of the spirit of the city even as a photograph expresses the character of the individual, but the city itself is more than a mere expression. I have often compared the difference between the modern city and the village from which it was developed with the difference between the modern battleship and the wooden frigate of Nelson's day. The latter was driven by the winds of heaven, the sails were set and furled, and the anchor was weighed by hand. The battleship of to-day is a bundle of nerves and in its hull is installed the most intricate machinery of many kinds. The crew knows little of the things that sailors of old had to know, but now includes specialists of many kinds, skilled in the use of steam and electrical machinery and the radio, and experts in scientific gunnery and navigation. Practically all the activities of a large town are represented on the battleship and its personnel includes opticians, dentists, physicians and surgeons, carpenters, painters, barbers, tailors, printers, even the ministry is represented. Its officers have been taught and drilled in almost every science as well as in the principles of local and international law.

The homes and offices of every modern city are equipped with electric lights and telephones and the cooking is done largely by gas, piped from a central plant. Electric cars and vehicles propelled by gasoline-driven motors take the people to and from their homes. Buildings, many stories in height and equipped with elevators, have taken the place of the old low-lying buildings. Detached dwellings are rapidly being replaced with apartment houses, each one sheltering many families. The streets are well paved and lighted and are cleaned more often and much better than they were formerly. Probably there are still as many horses in the streets as there were formerly, yet horse-drawn vehicles have been so largely supplanted by motor trucks of much greater capacity that it seems as if the horse is disappearing.

The unsanitary cesspools have been superseded by sewers which automatically carry away the wastes and the town pump has been supplanted by a wholesome water supply brought from distant sources. The newspapers are supplemented by motion pictures and by the radio, which give the people in their theaters and homes the news of the day. Religious, educational, hospital, and amusement facilities exist in great numbers, as a part of the equipment of the modern city, and are available for the poor as well as the rich to a far greater extent than ever before. The home of the laborer is fitted with comforts and conveniences that the wealthy man of the past could not command because they did not then exist. The general health of the people is better than in the more primitive days of the past. The comforts

of yesterday have become the necessities of to-day and what were formerly regarded as luxuries are now demanded as comforts.

The citizen has to pay, of course, for all these modern facilities and this fact is brought to mind forcibly by some statistics which I saw recently in a copy of the "Gazeteer of the State of New York" published in Albany in 1813. The population of the City of New York, according to the Census of 1810, was given as 96 373, including 1 686 slaves. The budget of the City for 1812 gave the receipts as $1 012 460.38 and the expenditures as $953 736.04. Included in the receipts was an item of $4 969.55 for street manure. The authorized budget of the City for 1925 is nearly $400 000 000 for an estimated population of about 6 000 000. In other words, while the population has increased 62 times, the budget has increased nearly 400 times.

Seldom do we pause to think of the enormous power which has been harnessed and is available within the confines of a single large city. It is estimated that the potential power transmitted through the water, gas, and steam mains, and the electric power cables, underlying the streets of New York, is at least three times the amount now being generated by all the hydro-electric plants at Niagara Falls on both sides of the river. The capacity of the central electric generating plants in New York alone is about 3 000 000 h. p. I have been informed that the condensing water pumped for these plants is eight times the quantity of water supplied to the city by its aqueducts.

In a material sense, the modern city is the result of the work which has been done by the pure scientist and the engineer. Without them, the city could not exist. To their skill and toil are due the water supply, the sewers, and the sanitary conveniences, the paved streets, the transportation facilities for local, suburban, and interurban use, the telephones, the electric lights, and all the many other facilities on which a city has come to depend for its very existence. Nor could the city be kept free from epidemics and its health be maintained at so high a standard without the excellent work of the sanitary engineer in co-operation with the chemist, the bacteriologist, and with medical research. So it has come about that typhoid and yellow fever have been largely eliminated and nearly all communicable diseases have been brought under control.

All the utilities which so largely affect our modern urban life require the services of specialists, carefully trained, each in his particular line of work. The day of the "Jack of all trades" has passed, because it is obvious that to-day, with such a tremendous amount of detail in every line of effort, one can no longer be an expert in everything and these experts are engineers by whatever name they may be called. To think of the long and patient study which has been, and is being, given to each one of these problems, to realize that the execution of the work is frequently done under the most discouraging conditions, and that progress is generally made in spite of and not with the help of the multitude, makes us feel the extent of the debt we owe to those who have labored in the laboratory and the designing room, the office, the shop, and the field, to accomplish results we accept so complacently. Few of

these men are known to the public because their deeds are not of the sensational character that commands public attention and most of them are too modest to expect any public commendation for their work.

The change to urban life has affected deeply the customs, the habits, and the thoughts of the people. In the simple days during the early life of the Republic, when men lived generally distant from one another, they were largely individualists, believing with Thomas Jefferson that government was a necessary evil and that they should get along with as little of it as possible, depending on their own efforts for success. Modern concentration of vast numbers of people in cities has led to the organization of industrial corporations with hundreds and frequently thousands of employees working under the same roof. Diversities of custom are disappearing. Organizations based on class consciousness are formed with the idea of bettering the material condition of particular classes of workers. More government is demanded. There is a drift toward paternalism and socialism; a tendency to lean on the State or National Government for help out of all difficulties; to lose that Anglo-Saxon spirit of independence which formerly prevailed and which was the foundation on which the structure of our national life was built. The fear exists that these tendencies are going too far; that the worker is being made into a machine and that so much effort is given to developing his material side, that the moral and the spiritual sides are forgotten. While city workers have escaped the hardness of the farm life of the old days, they are paying for the ease of life in a loss of that touch of fundamental things which is so necessary for true happiness and for the full rounding out and proper balance of humanity. We have boasted that the percentage of illiteracy is much lower in our cities than in many of the rural districts, but have forgotten that education consists not only in the ability to read and write, but that its true purpose is the teaching of an understanding of the lessons which Nature teaches, to a reading of "the sermons in the rocks," and "to the dreaming of dreams of human progress and to ultimate happiness and contentment."

The price we have paid for the many advantages which life in the great cities has given us is a large one. The opportunities and advantages of urban life have not yet come to compensate humanity for the restful quiet of the open country, for the simple pleasures it affords, and for the spirit of introspection which it fosters. Their comforts and luxuries are to be enjoyed only at the expense of a certain softening of character. The joy of doing is marred by the prevalent feeling of unrest, and the ever mounting cost of living in cities is a cause of anxiety to those of moderate means. It is becoming increasingly expensive to bring foodstuffs and other supplies into large cities, principally because of the restricted terminal facilities which are, however, in part, the result of a narrow or selfish vision of the particular community.

Large cities have come to stay and will doubtless continue to exist as long as industrial conditions and human nature remain as they are and as long as gregarious man retains the desire to live and work where other men congregate. Recognizing this, the efforts of every good citizen should be directed

to making cities what they should be, by each giving of his best thought to improving the quality rather than the number of inhabitants. Many are giving thought to the proposition that it would be better for the well-being of the human race if the tendency it now displays toward urban life could be checked. It is a grave question whether the future rapid growth of cities should be encouraged. It is obvious that our cities cannot continue to grow until they include the entire population. There must come a time when their growth will be checked by the inevitable process of economic laws, if thinking men do not find some way to check it before that time arrives. It does not conduce to the well-being of the people to have cities become too large. It is not economically sound to have them do so. The cost of transporting the people workward and homeward, of bringing food to them, and of providing for them in other ways, adds a great load to an already burdensome tax not only on themselves but on the other people of the nation.

There would be less concern regarding the present and future condition of our cities if the standard of governmental growth had kept pace with that of the material growth of our cities, but it has not. Municipal government in our country still falls far short of being what it should be, but it is to be remarked that municipal government like municipal engineering is a new problem for the world. If cities had not grown so fast there would have been more time to work out this new problem. Rapid growth is not conducive to the development of the best in government. Centuries were required to develop stable systems of self-government for States and Nations with their scattered populations, while these large cities have existed but a few generations and the problems of their government are new. Then, too, the problem is complicated by the fact that in our own country, at least, the population lacks social homogeneity. The immigrant, because of the sudden change from his old standards and conditions and by reason of his inertia toward a new environment, renders the task more difficult of solution. The sociological problems which confront municipal government are staggering in their proportions. We must not lose faith if the many evils of municipal government are not corrected at once. It will require the best efforts of wise and honest men many long years to accomplish this and it is a tribute to those who have thus labored that present conditions are no worse.

I believe we are making progress in the right direction. We find discouragement when, after going upward for a time, we drop back into the trough of the wave, but I believe each trough is higher than the preceding one and the trend is ever upward. Advancement is possible only through persistent, intelligent, and co-ordinated effort on the part of all good citizens. To be both effectual and permanent, reforms in the body politic must come from within and I have faith that they will so come. From an experience in public work of more years than I care to remember, I believe the reason we are so far behind is the apathy of the so-called good citizen. Included in this category are those engineers who show no interest in the affairs of their city except to complain when things go wrong. All such are apparently content to let others run their civic affairs for them and when they do criticize their

public servants they are likely, because of a lack of knowledge of the conditions, to condemn the one who has served them well instead of the one who deserves their condemnation. By so doing they work against their own best interests. Because of this apathy, because of lack of knowledge of its own affairs, and because of the habit of jumping at conclusions, the public has the deserved reputation of being a hard taskmaster for those on whom it has conferred the responsibility of directing public business. Many are misled by the mouthings of demagogues and do not take the time to reason out the particular thing which may be under discussion. Frequently, investigations are made of the conduct of public departments, sometimes for partisan reasons and political effect. To those familiar with such matters it is known that such investigations are usually far from being thorough, that they bring out only one side of the case, and that their conclusions, usually based on prejudice, are often fundamentally in error. The inevitable result in such cases is that the faithful and honest official, innocent of wrong-doing, is censured and discouraged. His department is disorganized because nothing is more demoralizing than a sense of injustice done. Every honest man chafes under such conditions and most men are honest. But, worst of all, the body politic suffers in such a case because whatever faith it may have had, has been destroyed. If public opinion was sufficiently alert to demand that all such investigations be rigidly thorough and impartial we would find ourselves a long step in advance. That is the way in which I believe engineers would conduct such investigations. They would bring reason to bear on them and would assemble and analyze the facts before reaching a conclusion.

I have referred to the part the engineer has played in the upbuilding of the modern city and of the debt the public owes him, but if he has done all these wonderful things, he has assumed a responsibility for them and owes a debt in turn. We cannot bring something into being without assuming responsibility for its proper development and for the use that is made of it. After all, works of the engineer's creation are but the means to an end, not the end itself. They contribute to the well-being of humanity and man is more than a physical being. Surely, the engineer has a duty to perform in addition to the development and care of material things, wonderful as they are. I do not like to think that the engineer can see only the steel and masonry structure he designs and builds. I hope he has a vision of what it is built for as the architect of the Middle Ages had of the great cathedral he erected, and that the spiritual, moral, and æsthetic sides of life have a great value to him.

We are justly proud of what the engineer has done and perhaps we have boasted about and given too much thought to his achievements, leaving to others the concern over the grave social and political problems which confront us with those great concentrations of population in our cities. How may he help in the working out of these problems? First of all, as a good citizen, he must do his share in all branches of community work. If he does less he fails in his duty. The engineer is no longer the pioneer of the early days when his work called him to more or less remote sections to build canals or to the frontier of civilization to build railroads. Lacking a permanent

abode, there may have been an excuse for neglecting his civic duties in those days, but not in these. There is no frontier with us now and as the engineer has, generally speaking, acquired a settled residence, he should take part in the affairs of the community in which he resides. By this I do not mean that he should enter politics as that expression is commonly understood. There are many ways to help besides that and many are finding the way, serving, as some are now doing, on school boards and other public and quasi-public boards to good effect and aiding in the solving of the multitude of vexing questions which arise. There is much civic work of a special nature that the broad-minded engineer is particularly qualified to assist in doing, such as the framing of building codes, zoning regulations, and of legislation affecting the regulation of public utilities.

It is not enough that he should understand his own problems. He should be able to explain them to laymen in language that is understood by them. Too often, I am afraid, the engineer fails to win approval of the authorities or of the public to a sound and meritorious proposition because he lacks the ability to translate his own sound thoughts into language that others who are not engineers will understand.

Do not let me be understood as claiming that engineers are superhuman beings, that they are made of better clay than others, or that they alone are responsible for all the good that has been accomplished. Like the rest of mankind they are not infallible. Others have been and are doing their duty to the world in their respective lines of endeavor; but it must be remembered that the engineer is trained to deal with fundamental things. He is accustomed to delve for the truth and to reject that which is unsound. Unless there is a foundation of truth in a proposition he will instinctively oppose it. Habit of thought inclines him to reason a thing out from cause to effect. Popular clamor and newspaper headlines do not sway his judgment. He is trained to look broadly and not parochially on all propositions involving the application of natural laws. Artificial political barriers do not appeal to him as they do to those who consider all things superficially, because he recognizes that the laws of Nature operate in the same way on each side of every State, National, or other political boundary.

For the engineer to bring these qualities to the public service he must in his civic associations be something more than the mere technician. I believe the engineer recognizes more than other citizens, and by force of example, by precept, and by teaching, he must show all men, that the days of waste are passing and the era of conservation has begun. With the waste must go all petty and partisan politics with so-called "log-rolling" and in its stead must be substituted constructive statesmanship to go hand in hand with the principles of conservation. If we are to live up to the ideals of our institutions our legislators must be leaders rather than followers. It has often been said that America is law-ridden and that its people have lost respect for its laws. A foreign critic said some time ago that we have more laws than all the other countries of the world combined and that we are the most lawless of all people. Is there not force in this criticism? Why should not engineers join with other

good citizens to correct some of the evils of the body politic we all recognize and which are due largely to the apathy of our people? The engineer can be a good citizen without losing his value as a technical man. He yields to none in respect and affection for the institutions of his country. His patriotism was shown by his work in the World War and in many other ways. Through active participation in civic affairs his vision would be broadened and the criticism that has so often been made as to the narrow outlook of the engineer on the affairs of the world would soon fall of its own weight.

As we read day after day the sensational matter that is called news and listen to the harangues of the popular orator and self-constituted regulators of the world we wonder if they represent the standard of the character and of the intelligence of our people. Should we so believe we could easily become discouraged as to the future of our institutions and even of civilization itself. Faith and courage will return if we realize that these are but manifestations of a small and vicious minority who bask in the sunshine of publicity. The great mass of all thinking men do their work quietly and without ostentation wherever right and duty call. In industrial plants, railroads, public works, schools and colleges, churches, hospitals, and on the farms quiet thinking men are doing the real constructive work of humanity and of civilization. With an instinctive reliance on and faith in the integrity of human nature, and believing in the permanency of those institutions of man which make for the advancement of knowledge and good of the world, they make their daily sacrifice to duty. They are representative of that divine force of progress which is irresistible in action because it is based on truth, on reason, and on character. Cannot some way be found to employ this force in the interest of civic betterment? All these workers are of the company of the engineer. With him they must advance to wider fields of greater effort, of greater promise, and of even greater service.

At the Centenary Celebration of the founding of the Franklin Institute, Dr. Arthur D. Little delivered an address on "The Fifth Estate" which he described as being "composed of those having the simplicity to wonder, the ability to question, the power to generalize, the capacity to apply. It is, in short, the company of thinkers, workers, expounders and practitioners upon which the world is absolutely dependent for the preservation and advancement of that organized knowledge which we call Science." While those who live in this estate may have the capacity to apply, we see it demonstrated on every hand that they have failed to make use of their ability. My plea goes out to those who, having the capacity to do, have failed to perform. To those of you who have not read Dr. Little's masterly address, fascinating in the beautiful simplicity of its language, I would commend it, and I know if you read it once, you will read it again. Quoting from it further:

"We see, in the ranks of science, knowledge without power, and, in politics, power without knowledge. An electorate, which regards itself as free, listens to the broadcast noise of manufactured demonstrations and is blind to the obvious mechanics of synthetic bedlam. The result is too often government by gullibility, propaganda, catchwords, and slogans, instead of government by law based on facts, principles, intelligence and good-will.

Thus, contrasting the difference between ability and accomplishment, engineers and men of scientific attainment must needs take to themselves the lesson that definite results obtained are a truer measure of the man than mere theoretical ability. The world will ever measure by performance. To it ability undemonstrated is synonymous with non-existence.

Thanks to the wisdom of our forefathers, the foundations on which our country's institutions rest are broad and deep. Incorporated in the design were high ideals and the superstructure was raised in fidelity and in faith. The task of its maintenance is that of each succeeding generation. As in the case of all structures built by man, repairs, renewals, and additions are necessary, but do not call for destruction of the original fabric. The skeleton of the structure must be maintained intact. The larger duty of the technical man calls him to this task. To the task he brings special qualifications. As a practical idealist his patriotism and civic spirit should be manifested not by a waving of flags and by boasts of superiority to others, but by searching out the defects in our present make-up and then by aiding others to raise the standards of civic affairs in the same deliberate and constructive way in which he designs and builds his physical engineering structures for the use and benefit of mankind.

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Paper No. 1573

SHERMAN ISLAND DAM AND POWER-HOUSE*

By H. de B. Parsons,† M. Am. Soc. C. E.

With Discussion by Messrs. E. E. Halmos, J. C. Meem, George A. Orrok, Lazarus White, Thaddeus Merriman, Henry Goldmark, Byron E. White, B. F. Jakobsen, Charles W. Comstock, Walter H. Wheeler, Jacob Feld, D. C. Henny, E. W. Lane, Howard M. Turner, Fred A. Noetzli, A. Streiff, and H. de B. Parsons.

Synopsis

This paper describes the design and construction of the Sherman Island Dam and Power-House on the Hudson River, about 8½ miles west of Glens Falls, N. Y. The development, built in 1921-23, is an hydro-electric plant with an ultimate capacity of 50 000 h.p. under a normal head of 66 ft. There are four 10 000-h.p. generating units now in operation; when additional regulation of the river is provided, the fifth unit will be installed.

The dam is 80 ft. 9 in. high and has a total length including the spillway and head-works of 2 069 ft. The lower half of the up-stream face is on a slope of 5 on 12, to take advantage of the weight of water on the dam in increasing the resistance to sliding.

Geologically, the site was affected by at least two heavy glacial actions, and a deposit of sand and boulders extends to great depth over a wide area. As a result, the development had many interesting features, both in design and construction. The main dam, of the multiple-arch type, was founded on sand and boulders with interlocking steel piles to form a cut-off. Pipes were placed through the base and the uplift pressure was determined. The spillway dam, horse-shoe shaped in plan, has a gravity section and is founded on rock. The head-race canal, 3 800 ft. long, was in fine sand and lined with

* Presented at the meeting of March 4, 1925.

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concrete. The power-house was built on very fine sand and considerable difficulty was experienced in excavating its foundation.

As there are few descriptions of dams and power-houses of this size founded on pervious material, a record of its salient features is given so that others may benefit. A further interesting feature is the measurement of the uplift pressure.

General Description

The Sherman Island Development, built for the International Paper Company between 1921 and 1923, is a 50 000-h.p., hydro-electric plant, on the Hudson River about 8½ miles up stream from Glens Falls, N. Y., and about 4 miles down stream from Spier Falls, N. Y.

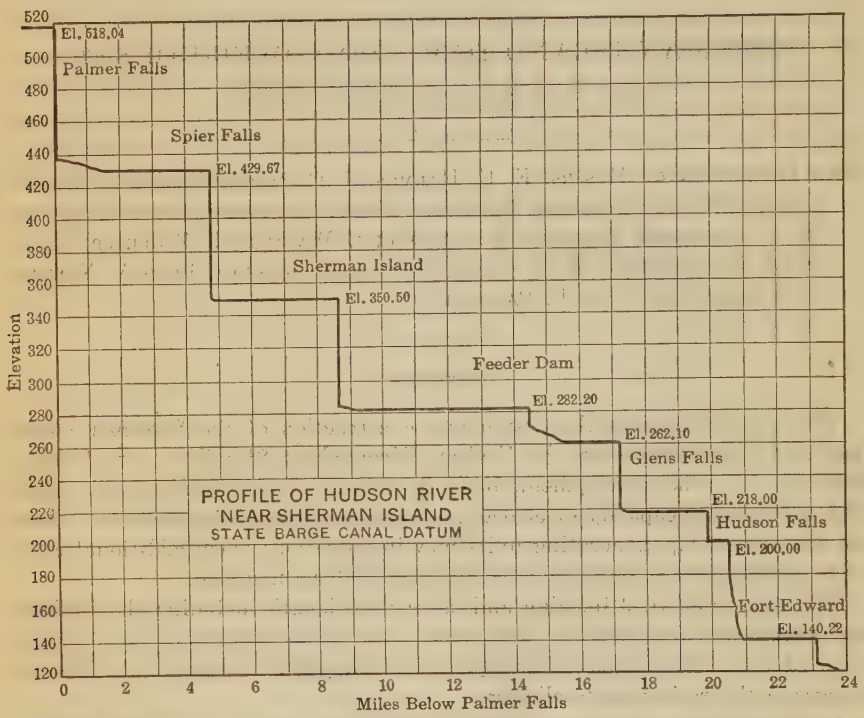


Fig. 1.

The profile, Fig. 1, shows the location of the Sherman Island plant with reference to other power developments on the river between Palmer Falls and Fort Edward, N. Y. The Paper Company owns the multiple-arch dam at Palmer Falls and the crib dam at Fort Edward, and is part owner of the multiple-arch dam at Glens Falls and of the recently built power plant at Feeder Dam. The Spier Falls Dam, owned by the Adirondack Power and Light Corporation, and the Sherman Island Dam were designed by the writer, who also was Consulting Engineer for the designs of the Palmer Falls and

Glens Falls Dams. The normal working heads are 84 ft. at Palmer Falls, 80 ft. at Spier Falls, 66 ft. at Sherman Island, 15 ft. 6 in. at Feeder Dam, and 46 ft. at Glens Falls.

The general plan for the development of power at Sherman Island was prepared by A. H. White, M. Am. Soc. C. E., Chief Engineer of the International Paper Company. The writer was asked in November, 1920, to report on the main and spillway dams, and he suggested cross-sectional designs for these structures, which were accepted by the Company. Mr. White then prepared designs for the remainder of the development (Fig. 2), worked out the details, and supervised the construction.

The original plan called for a power-house at the north end of the main dam. To secure the additional head due to the rapids below the dam, a tail-race was proposed through the channel between the north bank of the river and Sherman Island (Fig. 2), flood waters being kept out by a dike from the dam to the Island. The International Paper Company later changed the power-house location from the main dam to a site in Little Bay about 3 800 ft. down stream, and joined the head-works and intake dam at the forebay with a head-race canal, thus avoiding excavation of a tail-race channel in the river bed. The plan as finally adopted is shown in Fig. 2.

A contract was awarded to the Parklap Construction Corporation, and construction work was started in January, 1921, and progressed without interruption until completion. In October, 1921, the writer was retained as Consulting Engineer for the development when both the main dam and head-race canal were partly constructed. Water was turned on the wheels and current delivered in September, 1923, or two years and nine months after construction commenced. The total cost was about $7 000 000.

Site.—The site selected for the main dam was at the mouth of a gorge formed by the foot-hills of the Adirondack Mountains. Just below, there was a series of rapids, as the Hudson passed Sherman Island and then flowed out on a broad valley of sand deposit. Eleven borings were made at the dam site, the deepest being 80 ft. below the river surface. When the lower power-house location was selected, seven additional borings were made in its vicinity, the deepest being 90 ft. below the river. The character of the sub-surface material is indicated in Figs. 3 and 4.

General Dimensions.—Following are some of the statistics of the project:

Length of main dam	551 ft.	0 in.
Maximum height of main dam	80 "	9 "
Length of north wing-wall	174 "	6 "
Length of head-works structure	194 "	0 "
Height of head-works structure	42 "	0 "
Length of spillway on crest	864 "	0 "
Maximum height of spillway dam	46 "	0 "
Minimum height of spillway dam	20 "	0 "
Length of spillway gate structure	68 "	11 "
Length of training wall	217 "	0 "
Maximum height of training wall	26 "	0 "
Length of head-race canal	3 800 "	0 "
Width on bottom of canal	32 "	0 "
Width on top of excavation for canal	190 "	0 "

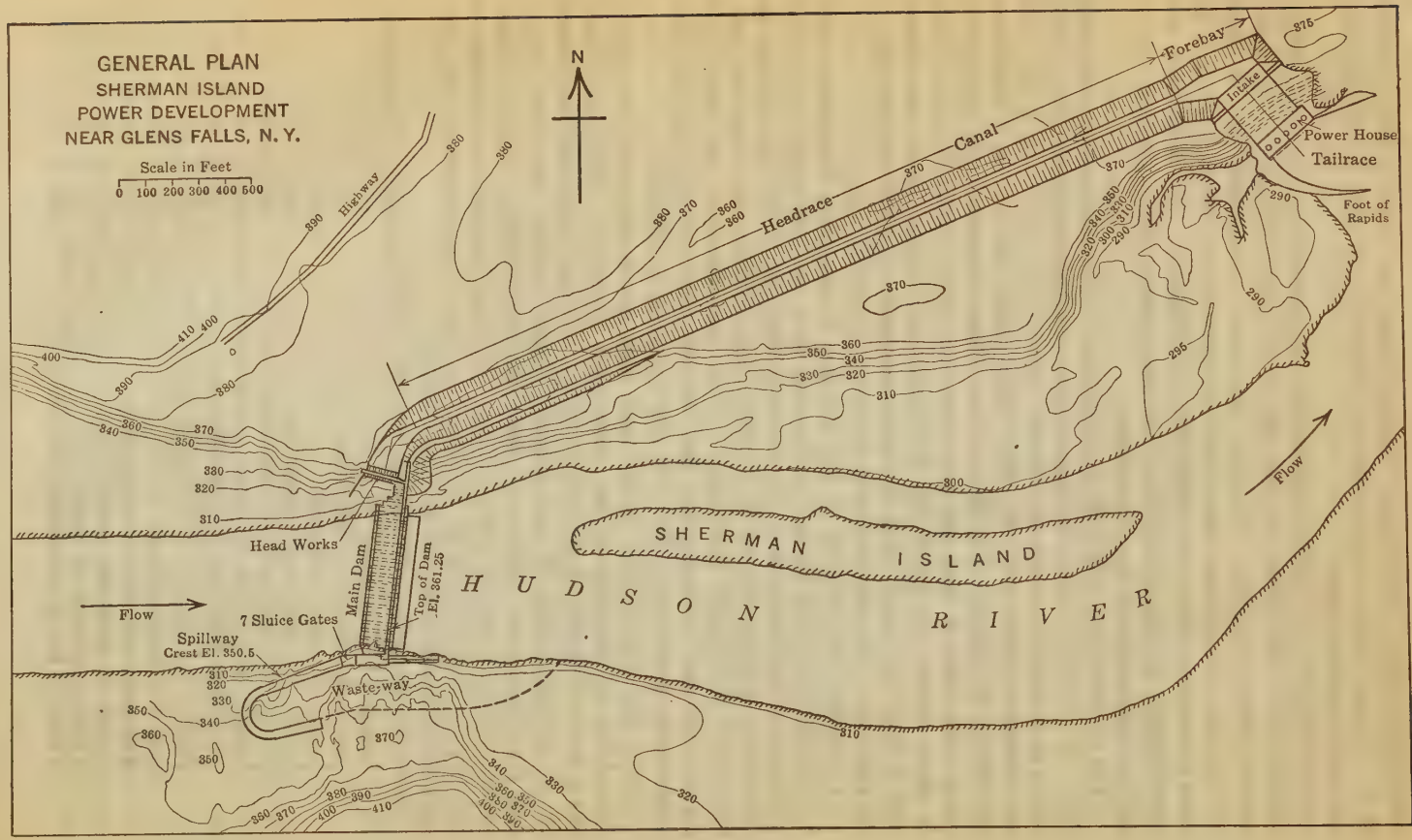

GENERAL PLAN
SHERMAN ISLAND
POWER DEVELOPMENT
NEAR GLENS FALLS, N. Y.
Scale in Feet
0 100 200 300 400 500
N
Highway
Headrace Canal
Forebay
Intake
Power House
Tailrace
Foot of Rapids
Head Works
Main Dam
Top of Dam El. 361.25
7 Sluice Gates
Spillway
Crest El. 350.5
Waste-way
Flow
Flow
SHERMAN ISLAND
HUDSON RIVER

FIG. 2.

Depth of excavation for canal	47	ft.	0 in.
Depth of water in canal (normal)	26	"	0 "
Length of intake (forebay) dam	200	"	6 "
Height of intake dam	49	"	0 "
Length of intake dam and wing-walls	349	"	9 "
Length of penstocks	176	"	1 "
Length of substructure under power-house	228	"	2 "
Width of substructure on bottom	79	"	0 "
Depth of substructure below original surface	21	"	0 "

Local Geology

Rock outcropped at the water edge on both banks and extended up the hillsides with little covering. In the plain-like area down stream from the site, rock is not found for about 5 miles (near Feeder Dam). Foundation conditions change at the dam site, there being two entirely different sets of geological formations. A fault crossed the development between the main dam and the upper end of Sherman Island, on a line practically parallel to the dam. Rock exposed in the canal excavation just behind the head-works on the north bank and in the waste-way channel on the south bank, exhibited fractured and twisted strata caused by this slip. The part broken off is buried at unknown depth under a sand deposit.

The geologic floor is the crystalline rock of the Adirondack Region, of complex origin and exhibiting great variability; it is commonly called Adirondack gneiss, and its interstices are filled with fused quartz. It has been worn and eroded by at least two glacial periods of prehistoric times, and plainly shows ice markings and scratches; it is hard, durable, and altogether suitable for foundation purposes.

The geologic rock floor in the river bed has been eroded and covered with glacial drift, solidly compacted, its upper layer containing boulders and cobbles embedded in sand and gravel. These boulders vary in size from ordinary cobbles to about 1-yd. stones, and are smooth and water-worn. Many are of foreign origin, probably carried down by the slow movement of the ice from remote regions; their deposit evidently marks a ground moraine. A section at the dam site is shown in Fig. 3. The pavement of larger boulders, chiefly confined to the upper level, is approximately 15 to 20 ft. in thickness, and forms an important element in supporting the main dam. The compacted material beneath, offers obstruction to water passage under the dam, and this "till" has been found satisfactory for foundation purposes.

The depth of the gorge at the main dam site was so great that it was not feasible to carry the structure down to the rock floor. However, both the spillway dam and the training-wall for their entire lengths, and both ends of the main dam rest on bed-rock of firm character.

Down stream from the fault the crystalline rock floor is lost. It must exist, but at what depth is not known. The great sand deposits in the plain-like area are the "scour and fill" material from water and ice workings of bygone ages. Apparently, the outwash sands of the valley come against the margin of the crystalline floor rock at the fault line, near the dam, without any underlying till or transition material. The post-glacial action is shown in the stream

erosion of the finer materials, leaving the boulders as a pavement along the river bed. This boulder pavement, placed by Nature, has exhibited resistant qualities against dislodgment and scouring by floods of centuries. At Sherman Island, the obstruction was sufficient to cause the river to cut channels on either side, and leave a high area which now forms the island.

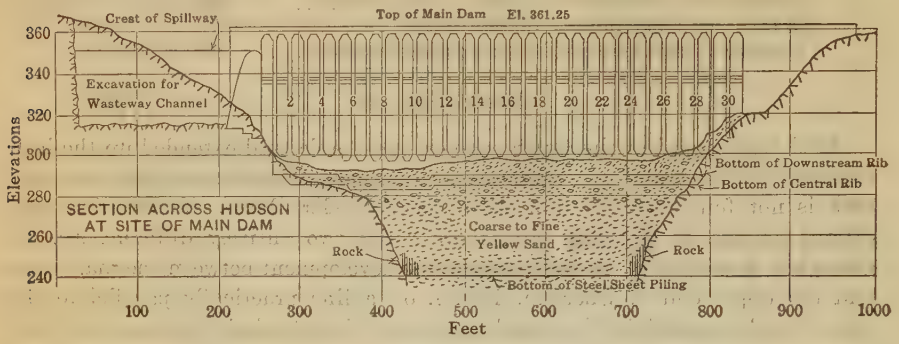


FIG. 3.

Along the head-race canal the upper layer is a cobble drift, several feet in thickness, probably a residuary ground moraine. Next below are cross-bedded outwash sands, having approximately horizontal beddings; many of the layers, however, are lenticular in form. The sand varies in size from moderately coarse to very fine, and "ripple" marks were frequently uncovered. Beddings of fine sand, generally dark in color, containing some aluminum and resisting water were found at vertical intervals.

At the power-house site, the upper layers of fairly fine to coarse sand are underlain by very fine deposits, which are in the nature of silt and con-

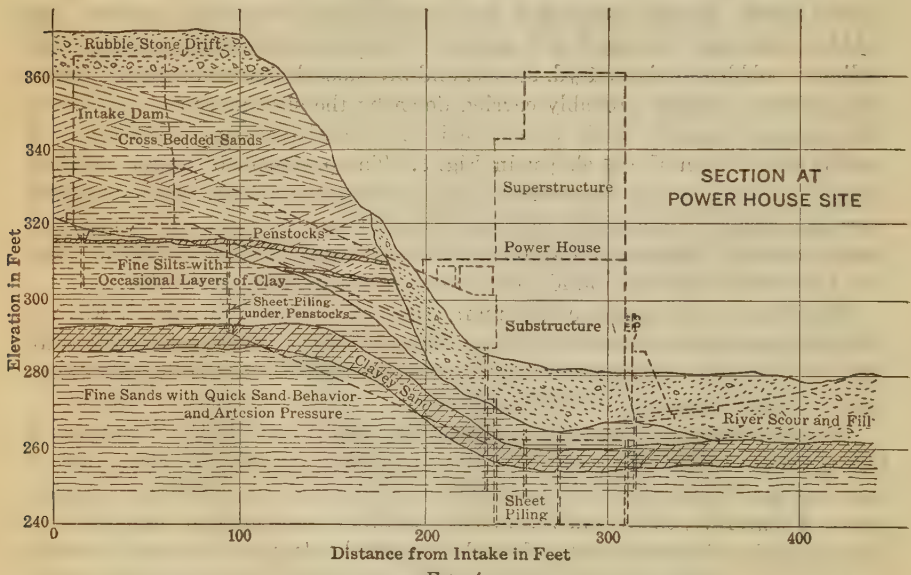


FIG. 4.

tain enough clay to change entirely the behavior of the material. Beneath these fine sandy silts, the layers of which are impervious when undisturbed, is a deposit of very fine sand of apparently great depth, as exploration borings did not reach the bottom. Beneath this fine deposit of almost pure quartz sand must be a firm ground moraine. The materials at the power-house site, as outlined by Dr. Charles P. Berkey, the geologist, are sketched diagrammatically in Fig. 4 on which is also indicated the power-house, the intake dam at the forebay, and the penstocks.

The depth and character of the fine sand at the power-house site prevented the use of any substructure supports. In consequence, the base was kept above the fine sand layer and did not penetrate (except the steel sheet-piling) the blanket of sandy silts and clays, which cap the finer materials. As the borings which entered the "fine sands with quick behavior" exhibited an Artesian pressure, it seemed evident that the blanket or capping of clayey sand was practically impervious, and that the fine sands would not be released unless disturbed. The Artesian effect was due to the general water-table standing in the bank at elevations above the river level, thus furnishing a "head".

After this Artesian pressure was noted, a search was made for a more favorable power-house site, but all the territory in the neighborhood was found to be of similar character.

Hydrology

The drainage area above Sherman Island is 2 782 sq. miles. The region is mountainous, part of it heavily wooded, and contains many lakes and ponds. The ground is rocky or only lightly covered. The principal tributaries above the dam site are the Indian, Schroon, and Sacandaga Rivers. The average rainfall on the Upper Hudson water-shed probably exceeds 42 in. per year. The U. S. Geological Survey has recorded the flow for many years at gauging stations at Hadley and at Spier Falls. The normal flow, being an average of twenty years, 1901 to 1920, inclusive, is shown in Fig. 5, together with the regulation from Indian Lake storage, and, by way of comparison, the minimum flow during the dry year of 1923.

The greatest known flood, 89 100 sec-ft., was recorded on March 28, 1913. For the last eleven years, the maximum discharges* at Spier Falls (or Hadley) have been, as follows:

Date.	Discharge, in second-feet.
March 28, 1913	89 100
April 20, 1914	52 000
April 13, 1915	26 600
May 19, 1916	28 000
June 12, 1917	38 100
April 4, 1918	34 500
April 13, 1919	32 000
April 1, 1920	29 000
March 22, 1921	32 800
April 13, 1922	58 000
April 8, 1923	36 400

* Reports of New York State Engr., and U. S. Geological Survey.

Maximum flows are traceable to heavy snow accumulations on the upper water-shed and a rise in temperature accompanied by warm rains. A study of the flood flows during the past twenty years, platted on probability paper, would indicate that a flow greater than the recorded maximum is not likely to occur oftener than once in about 100 or 150 years. During this twenty-year period, the maximum 24-hour flood discharge was 84 200 sec-ft., on March 28, 1913, and the minimum, 16 500 sec-ft., on May 3, 1911.

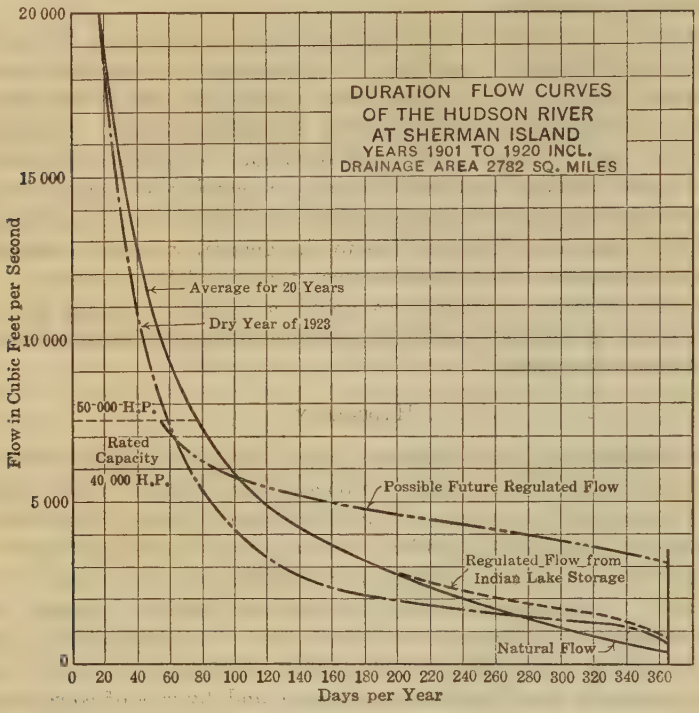

DURATION FLOW CURVES
OF THE HUDSON RIVER
AT SHERMAN ISLAND
YEARS 1901 TO 1920 INCL.
DRAINAGE AREA 2782 SQ. MILES
Flow in Cubic Feet per Second
20 000
15 000
10 000
5 000
0
Average for 20 Years
Dry Year of 1923
50 000 H.P.
Rated Capacity 40 000 H.P.
Possible Future Regulated Flow
Regulated Flow from Indian Lake Storage
Natural Flow
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360
Days per Year

Fig. 5.

The flow of the Hudson River is regulated somewhat from the Indian Lake storage. It is proposed to increase this by additional storage reservoirs on the Upper Hudson, Schroon, and Sacandaga Rivers. This regulation will augment the flow during the low-water period of the year, as seen in Fig. 5, and will reduce the maximum flood discharges materially. Thus, the great flood of 1913 would have been reduced from 89 100 sec-ft. to about 55 000 sec-ft. and other floods in proper proportion.* Great public benefit will result from this, as the Hudson is destined to play an important rôle in the commercial and industrial life of New York State.

The lumber industry on the upper river is not active, although a large number of logs still float down during high-water stages. Perhaps, later, log driving will be under control and some system of river cleaning will be established.

* Report on the Water Power and Storage Possibilities of the Hudson River, New York Water Power Comm., 1922.

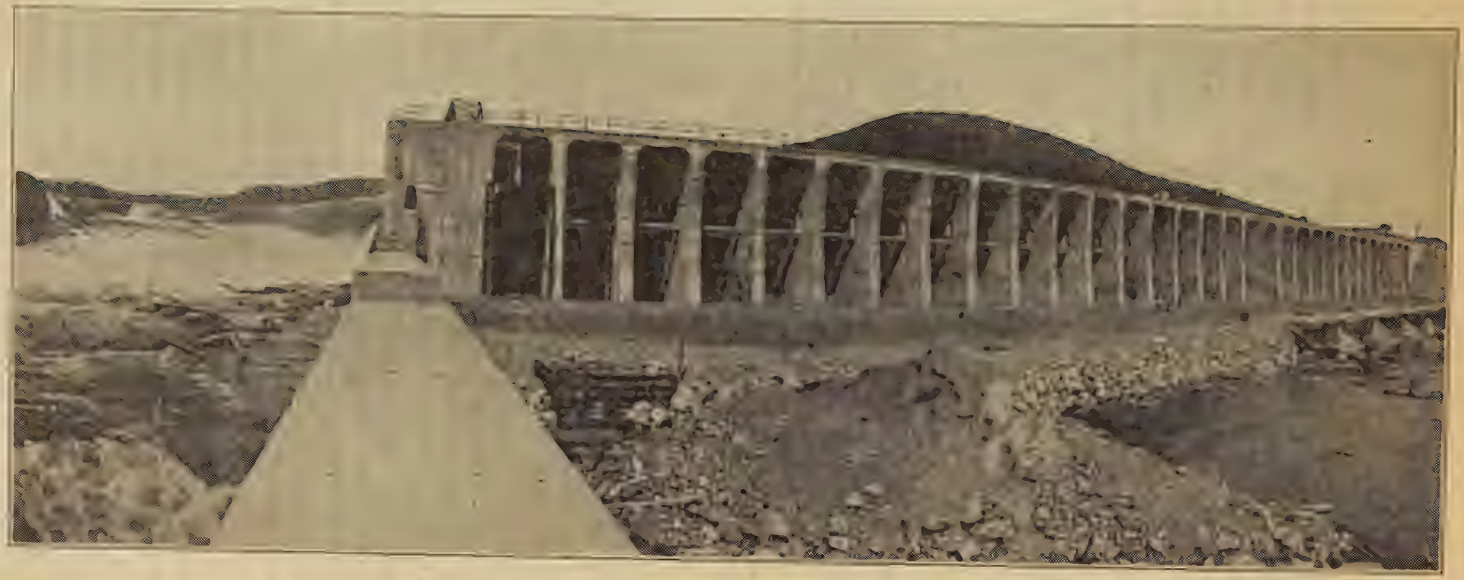


FIG. 6.—VIEW OF SHERMAN ISLAND DAM AND WASTE-WAY FROM DOWN STREAM.

Main Dam

The main dam, Fig. 6, is of reinforced concrete, the ends being on rock and the central portion on a sand and boulder deposit. The south end butts against the spillway dam, and the north end forms a wing-wall, which joins the head-works built at the entrance to the head-race canal (Fig. 2). The dam is straight, although the writer's original plan contemplated a slight camber up stream. It is a multiple-arch structure, having thirty-one bays, each 19 ft. center to center of buttresses. The length, including the north wing-wall, is 725 ft. 6 in., and the maximum height, 80 ft. 9 in., as measured from the under side of the central rib at the lowest part to the walk on top.

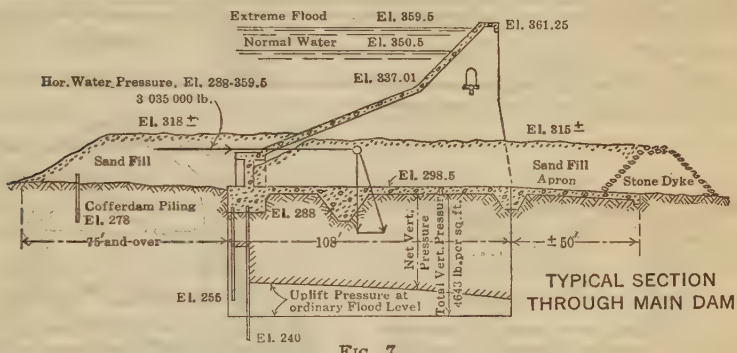


Fig. 7.

The buttresses, 3 ft. 6 in. thick, stand on a concrete base, 108 ft. wide by 3 ft. thick (Figs. 7 and 8), which extends from rock wall to rock wall across the river. The lower portion of the up-stream face is inclined 22° 40′ (5 on 12) to the horizontal, and the upper portion, 45 degrees. At the up-stream corner or heel, openings 10 ft. high by 13 ft. 6 in. wide were left in each bay (Fig. 8) for passing the river prior to closing the dam.

There are three concrete ribs under the base at the toe, the heel, and near the middle to prevent any tendency to scour, and to increase the resistance to sliding. Two rows of Lackawanna steel sheet-piles, with interlocking joints, were driven under the up-stream edge. The piles in the first row were 55 ft. long, of 14-in. arch web, and weighed 40.83 lb. per lin. ft., except about forty of 12¾-in. straight web, which weighed 37.187 lb.

After the first row was driven to Elevation 240, a second row 40 ft. long was driven 6 ft. up stream to Elevation 255. Where the rock rose at the banks, piles were driven to form box coffer-dams, and the main row joined into the ends, as shown in detail in Fig. 9. These end piles were driven hard against the rock, so as to close all openings. At the south abutment, the pile ends were cut diagonally to fit the rock slope, and the webs were split so that the ends would crumple up and make a tighter closing. As much sand as possible was dug out of these pile coffer-dams, and the space filled with concrete. Grout pipes were used where the sand could not be removed.

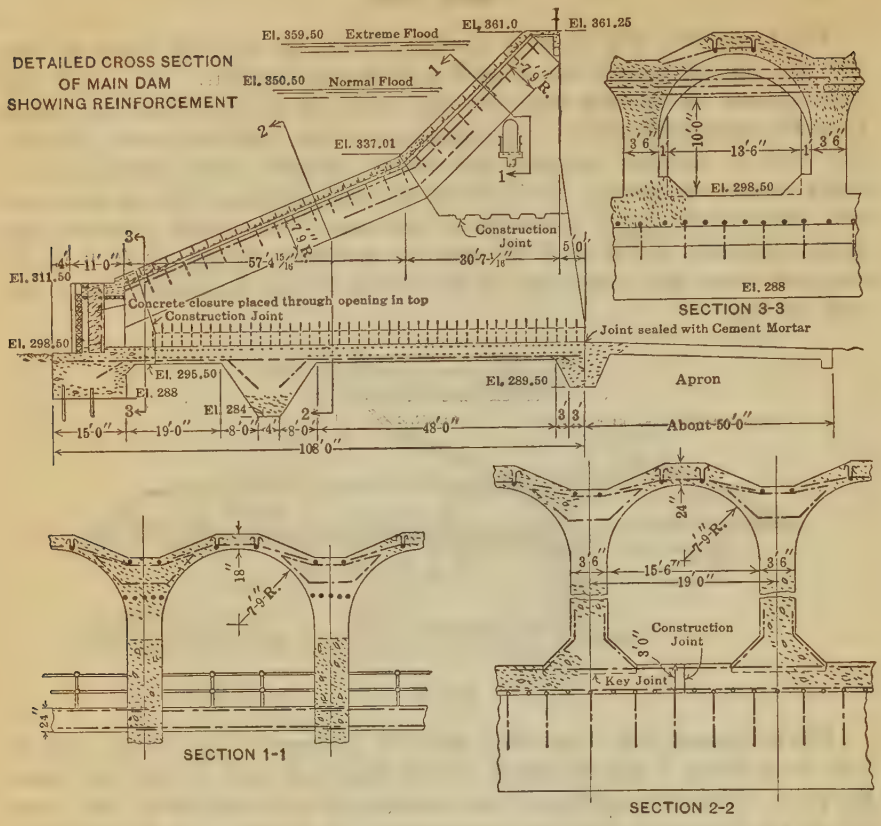
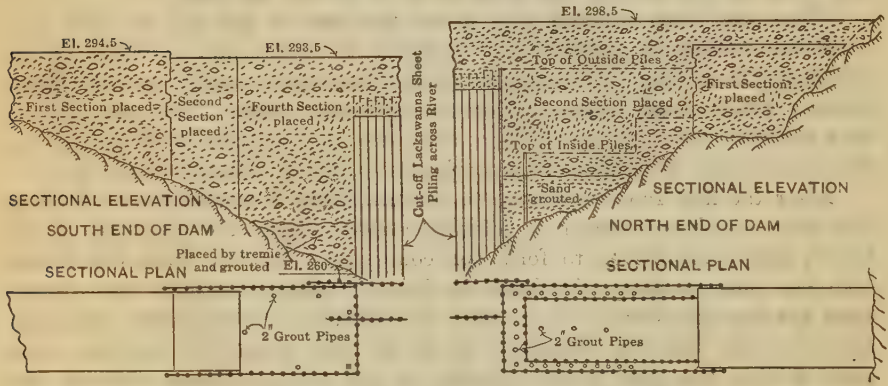


FIG. 8.



JUNCTION BETWEEN SHEET-PILING AND ROCK AT END OF DAM

FIG. 9.

The piles were driven by a steam hammer having movable parts weighing about 1 250 lb. The number of blows required for a penetration of 1 in., or less, when they were driven to final position, were as given in Table 1.

TABLE 1.—Pile-Driving Data.

	First Row.	Second Row.
	55 ft. long.	40 ft. long.
For Piles All in Sand :		
Maximum blows	104	92
Minimum blows	6	7
Approximate average	25	15
For End Piles, with Foot on Rock :		
Maximum blows	150	66
Approximate average	25	50

At the start, each pile was driven singly, with the first pile near the middle of the row; afterward, they were driven in "gangs", and greater success was obtained in keeping them straight and in lock. When a "gang" of, say, ten or a dozen piles was partly driven, another "gang" was set up and driven. Then, the first "gang" was driven a little more. This operation continued until the piles were down to the required elevation. Special wedge-piles were driven as required, to keep the row straight and vertical.

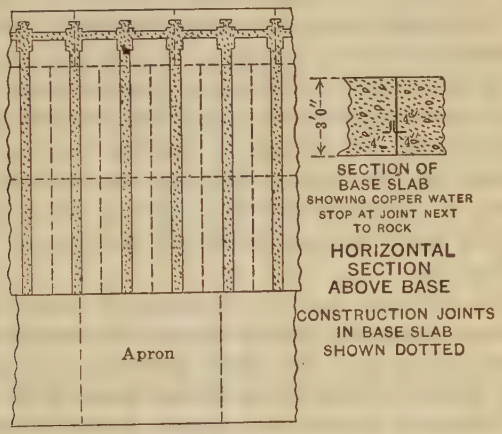

3'0"
SECTION OF BASE SLAB SHOWING COPPER WATER STOP AT JOINT NEXT TO ROCK
HORIZONTAL SECTION ABOVE BASE
CONSTRUCTION JOINTS IN BASE SLAB SHOWN DOTTED
Apron

Fig. 10.

Before pouring the base concrete, all loose surface material was removed from the river bed, but the solid packed portion and boulder layer were disturbed as little as possible. Under the south portion of the dam the base was made at a lower elevation in order to fit the contour of the river bed. The base was reinforced, the rods projecting across the construction joints (Fig. 10). Where the base passed from rock to sand foundation, the rock was trimmed or cut, and a tapering sand cushion placed so as to ease the transition and minimize settlement troubles as much as possible.

The buttress concrete was poured continuously between the construction joints shown in Fig. 8, leaving the arch decking to be placed afterward. The buttresses and decking were reinforced with deformed rods, varying in size from ½ in. to 1¼ in. The buttresses were located at 19 ft. centers, which close spacing made the loading on the base slab practically uniform. The comparatively short span, moreover, facilitated the repeated use of the arch forms, which were easily handled by the overhead cableway. For simplicity in form work, the thickness of the buttresses was kept uniform. An inspection gallery extends through the dam (Fig. 8), its reinforced concrete walk acting as a strut between the buttresses.

The arched barrels of the up-stream decking are circular with a radius for the intrados of 7 ft. 9 in., measured perpendicularly to the slope. Each extrados (Fig. 8), was made three-sided, flat on top, and inclined on both haunches. This flattening simplified the form work and the labor of pouring the concrete. The deck is 24 in. thick in the lower part, and 18 in. in the upper. Before the dam was closed, the northern half was given four coats of "Ferritex" water-proofing.

The arched deck had a flat incline so that the superimposed weight of water would increase the resistance to sliding. To add still further stability against sliding, the interiors of the bays between the buttresses were partly filled with sand. For each 19-ft. bay the water on the dam to the extreme flood level weighs 3 004 700 lb., the concrete, 4 272 000 lb., and the sand fill within the dam (exclusive of the apron), 2 250 000 lb., making a total of 9 526 700 lb. This weight gives an average loading of about 4 643 lb. per sq. ft. The resultant cuts the bottom of the base about 1½ ft. from the center. Under normal water conditions, the resultant cuts the base slightly on the up-stream side of the center, while for ordinary flood levels, it is practically at the center and produces a uniform loading. This downward pressure is reduced by the uplift, which was measured by test pipes built through the base, as described hereafter. Taking actual test readings for ordinary flood-pond level, the magnitude and distribution of the pressures are shown on Fig. 7.

An important provision was the dumping of about 30 000 cu. yd. of fine sand up stream along the dam to improve the seal of the cut-off sheet-piling. The top of this sand was about Elevation 318, that is, the dump was about 20 ft. high and extended up stream about 75 ft., or more. It was considered that this sand blanket was needed, as the Hudson deposits little sediment on account of the close proximity of the dams above the site. In front of the roller gates, described later, the sand was omitted until the final closure.

Settlement and Alignment.—Observations were made to record the settlement and change in alignment as the buttresses and arches were poured, as the fill was made, and as the water rose in the pond. Table 2 gives the cumulative settlement and down-stream movement for Buttresses 6 to 26. End Buttresses 1 to 5 and 27 to 31 were wholly or partly on rock so that the movements of these buttresses were small.

The cross-piling between the north and south coffer-dams was under Buttress 12. When construction started on the south portion, the adjacent buttresses in the north portion were affected. Table 2 gives in brief the changing conditions. It will be noted in general that both settlement and down-stream movements increase gradually toward the center of the dam; the maximum settlement being 1.4 in. at Buttress 14, and the maximum horizontal movement, 1.63 in. at Buttress 18. The survey line for alignment was along the walk on top of the dam. Any uneven settlement or elastic deformation of the structure, therefore, would be reflected in the observed alignment.

Surveys taken since those recorded in Table 2 have shown little change in either settlement or alignment, and the dam appears to be at rest with the exception of changes due to temperature.

TABLE 2.—Movement of Main Dam.

Date.	Progressive Settlement, in Inches.											Remarks.
	South.			Buttresses.							North.	
	6.	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.	
October, November, December, 1921.	....	....	....	0.33	0.54	0.31	0.29	0.33	0.37	0.36	0.26	Construction in north coffer-dam.
September, 1922.	....	....	....	0.33	0.55	0.33	0.30	0.37	0.53	0.45	0.26	Concrete finished, Buttresses 13 to 30.
December, 1922.	0.05	0.00	0.00	0.57	0.61	0.52	0.44	0.42	0.53	0.45	0.26	North coffer-dam removed. South coffer-dam closed. Fill started under north arches.
February, 1923.	0.14	0.14	0.15	0.90	0.89	0.82	0.70	0.59	0.70	0.64	0.44	Pond raised to about Elevation 335.
March, 1923.	0.24	0.29	0.35	1.13	1.10	0.99	0.89	0.80	0.86	0.80	0.53	Pond raised to about Elevation 350.
June, 1923.	0.40	0.45	0.45	1.24	1.40	1.29	1.08	0.90	1.05	1.00	0.76	Filling under arches completed.
September, 1923.	0.40	0.45	0.45	1.24	1.40	1.29	1.08	0.90	1.05	1.00	0.76	Pond had been over Elevation 354.

Lateral Movement, in Inches, After Dam Was Finished, as Surveyed Along Walk on Top.

Date.	South.			Buttresses.								North.		Remarks.
	4.	6.	8.	10.	12.	14.	16.	18.	20.	22.	24.	26.	28.	
January 17, 1923.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	Datum taken. Temperature 22° Fahr. Dam closed, March, 1923.
April 10, 1923.	0.13	0.36	0.61	0.74	0.74	0.74	0.82	0.90	0.99	0.86	0.73	0.62	0.24	Temperature, 52° Fahr.
April 19, 1923.	0.25	0.48	0.78	1.04	1.03	1.16	1.10	1.14	1.13	0.96	0.86	0.70	0.30	Sand filling nearly completed. Temperature, 61° Fahr.
April 29, 1924.	0.30	0.55	0.97	1.09	1.13	1.45	1.52	1.63	1.56	1.31	1.33	0.96	0.42	After one year. Temperature, 72° Fahr.

Apron.—To increase the length of water travel under the dam and as a precaution against scouring during construction, when the river was being passed through the bay openings, the base was extended down stream as an apron 50 ft. wide (Fig. 7). Sand filling was placed on this apron to supply weight against uplift and to resist any sliding movement of the dam. A dike of broken stone protects the down-stream edge of the fill and concrete

apron against wash from back currents created by the discharge from the waste-way channel.

Spillway Dam and Waste Channel

The spillway dam has a total crest length of 864 ft. A maximum flood of 100 000 cu. ft. per sec. will cause an elevation of the pond surface of 8 ft. 9 in. above the crest. The top of the main dam would then be 2 ft. above the flood level without allowing for the flow through the head-race canal and the spillway sluice-gates. As the drainage area above the dam is 2 782 sq. miles, there is 1 lin. ft. of crest per 3.22 sq. miles. The area of the pond is 277 acres.

The spillway dam was founded on firm rock for its entire length. The ogee form in the writer's original design, shown by the dotted line in Fig. 11, was omitted in construction.

The dam was built in about 50-ft. sections, the concrete being poured in alternate sections with as few horizontal joints as possible. Right-angled grooves, 8½ in. deep, were provided to act as keyways between sections. The concrete was not reinforced.

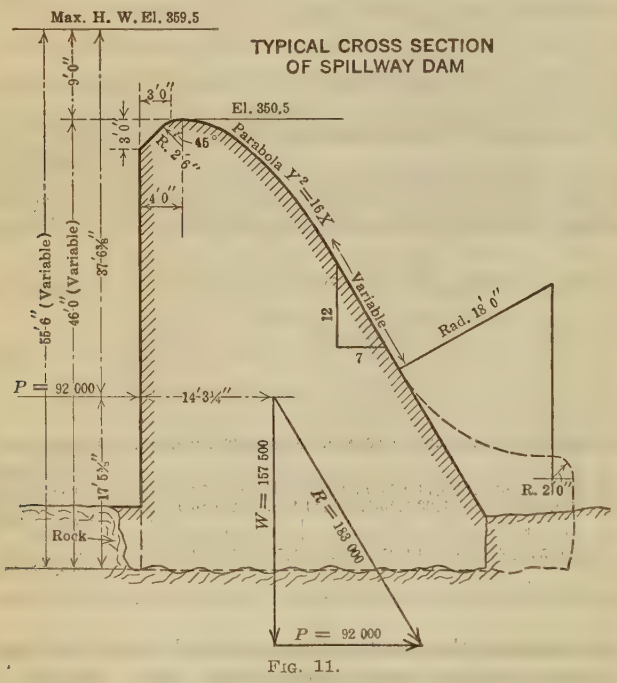


Fig. 11.

Brass pipes, 2 in. in diameter and 12 in. long, 5 ft. on centers, were placed along the crest, for the insertion of steel rods to hold flash-boards, 2 ft. high.

Seven gates to control the pond water were placed in a raised part of the spillway dam. Each gate is 13 ft. 4 in. high by 5 ft. 10 in. wide, made of

8 by 8-in. yellow pine timbers dressed and bolted to two 10-in., 54-lb. Bethlehem H-beams, which carry racks. The racks engage in a train of hand-power gears for raising and closing the gates. The wearing edges of the gates are protected by 4 in. by ½-in. bronze strips and seat against 6 by 4 in. by ¾-in. steel angle-frames anchored to the concrete.

A concrete training wall extends down stream, so that the discharge from the waste-way may not cause scour or dangerous back eddies along the front of the main dam; it has a gravity section, without reinforcement, and is founded on rock.

Waste-Way Channel.—The waste-way, excavated in rock, is 172 ft. wide on the bottom opposite the end of the main dam. The bottom was roughly graded to a slope of 3.5%, increasing to 5% at the upper end. During extreme discharges, the velocity of the waste water will be about 31 ft. per sec. and its depth about 18 ft. when passing the end of the main dam. The designing estimates for discharge and velocity were verified during the high-water season of 1923.

Head-Works

The head-works at the entrance of the canal has a concrete base, built on rock and supporting nine reinforced buttresses, each 2 ft. 6 in. thick. The base contains a number of 1½-in. holes in each bay, behind the gates, to relieve any uplift pressure when the canal is empty. One end of the structure joins the wing-wall of the main dam and the other abuts the rock forming the opposite side of the canal.

An interesting feature is the design of the gates, which are ten in number, each 9 ft. 6 in. wide. Each gate has three sections, of steel frames filled with concrete, reinforced with rods welded to the frames. The lower and middle sections are joined by short links with the joints caulked; the upper and middle sections are connected with slotted links which permit the joints to open about 12 in. on lifting. Water can thus enter the canal to relieve the pressure on the gate and facilitate the raising of the lower part. Each gate weighs about 25 tons.

The gates are operated by overhead electric trolleys, and are controlled from the power-house for quick closing.

Head-Race Canal

The canal was dug in sand, except the first 150 ft. at the head-works end, which was in rock. The section is 32 ft. wide on the bottom (Elevation 324), with 1 on 1.6 side slopes to the ground surface at about Elevation 375. The excavation was made to approximately the proper section, with a longitudinal bottom slope of 1 ft. from the head-works to the intake dam. One-half the bottom concrete lining was poured in sections, 16 ft. wide, 42 ft. long, and about 8 in. thick. The other half of the bottom was used for a construction track to convey concrete from the mixer. After the first half was finished, the track was shifted to the concrete and the other half poured. The side slopes were then trimmed by hand scrapers to their true slopes, and the concrete was poured in rectangles 21 ft. long by 8 in. thick, to

Elevation 346. From a construction joint at this point to the top of the lining, Elevation 356, the concrete was 3 in. thicker. The rectangles were poured alternately. Two timbers, each 6 by 4 in., were laid on the slope at each side of a rectangle to be poured. Between them was placed a No. 24 gauge (B. & S.) copper strip, 8 in. wide, crimped into a 1-in. loop, so that the copper water-stop would project 3 in. into each slab. The joints were brazed. The top forms, in 2-ft. sections, were of strong construction in order to support the concrete weights placed on them to counteract the uplift of the fresh concrete. The reinforcement, consisting of expanded metal of 3 by 8-in. mesh, weighing about 1 lb. per sq. ft., was laid on the finished grade, and pulled up by hooks into the proper position as pouring progressed. Only selected aggregate of small sized stone was used. Intermediate rectangles were poured in the same way, except that the inclined timbers were omitted, as the adjacent concrete slabs took the place of side forms.

The joints were filled with a bitumastic preparation known as Bridge cement. This was removed subsequently and the joints were grouted with a cement gun. The lining was sounded after it was all in place; where voids were indicated, holes were drilled and the grout was forced in under pressure. The mix of the grout was 0.1 part lime, 1 part cement, and 2½ parts sand.

After filling and quickly emptying the canal, weakness was disclosed in the lining at the down-stream end, due to ground-water and leakage causing hydrostatic and earth pressure in the fine sand some of which showed "quick behavior". For about 1 100 ft. (as also in the forebay), the lining was strengthened by building the north slope in steps. Concrete struts (Fig. 12) were constructed over the bottom slab of the canal. As the strutting was not adaptable to the forebay, the bottom slab there was covered to a depth of about 3 ft. with 2 500 cu. yd. of broken stone, to weight it against any uplift pressure and to oppose any inward thrust at the toe of the side slope lining. The broken stone was grouted with a cement gun in front of the racks to prevent dislodgment. As an additional precaution, flap relief valves of composition metal were placed about every 21 ft. longitudinally. These valves were 4 in. in diameter and mounted on pipes projecting 9 in. above the bottom concrete.

The canal lining was built to Elevation 356. At the forebay, the lining was built to Elevation 362, so as to protect the banks near the intake dam from surge waves.

At normal operation, with the water at the crest of the spillway, the cross-sectional area of canal above the cross-struts is 1 924 sq. ft. With five turbine units running at full gate, the water demand is about 7 500 cu. ft. The maximum velocity, therefore, is 3.9 ft. per sec.

In order to observe ground-water conditions as well as to detect any excessive leakage through the lining, eleven 2½-in. pipe wells were driven along the canal and forebay banks to about the elevation of the canal bottom; also, two additional wells, one on each side of the penstocks near the power-house. Observations of the water surface in the wells on the banks of the canal and forebay show only slight variations from the ground-water levels

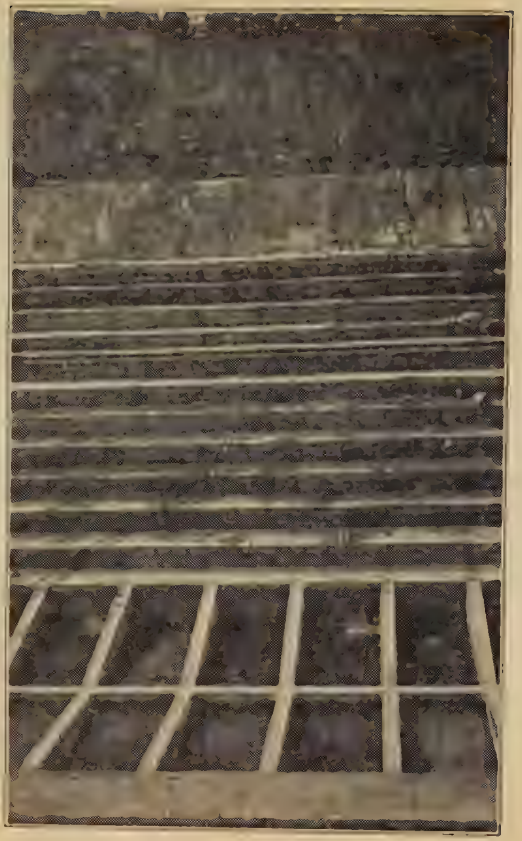


FIG. 12.—CANAL LINING STRUTS.

FIG. 13.—PENSTOCK FORMS, GATE-HOUSE IN BACKGROUND.

as determined during construction, indicating that leakage through the canal lining is small.

The water levels in the two pipe wells near the power-house show a variation of about 7 ft. These wells are at a much lower elevation than those along the canal or forebay, and this variation indicates changes in the ground-water at the lower level. This is probably effected somewhat by the obstruction to the flow of ground-water offered by the sheet-piling under the intake dam and by the penstock and power-house structures.

Intake Dam

The intake or forebay dam was founded in the sand of the plateau, as borings did not indicate rock within economical reach. After the sand had been excavated to grade, one row of Lackawanna interlocking steel sheet-piles was driven, as a cut-off, to a depth of 15 ft. below the bottom of the base slab; at the ends, where the wing structures tied into the sand, the piles were stepped up. The total length, including the wing-walls, is 349 ft. 9 in., of which 200 ft. 6 in. is the intake dam. The height is 49 ft., the width on the top, 50 ft., and on the bottom, 55 ft. The base projects into the forebay about 5 ft., making the total width of the bottom slab about 60 ft. This projection was made to increase the stability when the stop-logs are in place and the water openings through the dam are empty.

This dam, Fig. 14, has fifteen openings for water, that is, three openings for each turbine unit. Each opening is 10 ft. 0 in. wide between the dividing buttresses, which are grooved for stop-logs and Broome gates. At the east end there are two concrete-filled, steel-frame gates, each 8 ft. wide, controlling the log and ice chute.

Racks, of 4 in. by $\frac{3}{8}$-in. steel bars, spaced $3\frac{1}{2}$ in. apart, protect each intake opening. The trash collected is raked into a reinforced concrete trough whence it can be flushed with water into the log and ice chute. Steam pipes from a heating plant near the power-house are carried to the racks, to free them from ice if occasion should require.

A brick gate-house on top of the dam contains individual electric motors and hoisting gears for the Broome gates. The dam is wide enough for a standard gauge track alongside the gate-house, which track connects the power-house with the railway to Glens Falls.

The weight of the intake dam and gate-house, including the equipment and water, is about 2 tons per sq. ft. of base area.

Settlement and Alignment.—Surveys were made at intervals to determine the settlement and changes in alignment, with results as given in Table 3. The survey plugs were located on top of the dam, a few feet from the down-stream edge as follows: Plug 1 at the west end of the west wing-wall; Plug 2 at the west end of the intake dam, about 65 ft. from Plug 1; Plugs 3, 4, 5, 6, and 7 equi-distant along the intake dam, about 38 ft. apart with Plug 7 at the east end; and Plug 8 at the east end of the east wing, about 80 ft. from Plug 7. Plugs 1 and 4 were disturbed during construction.

Table 3 supplies the information required to study the actual movement of the dam. When the canal is empty, the resultant pressure is about 2 ft.

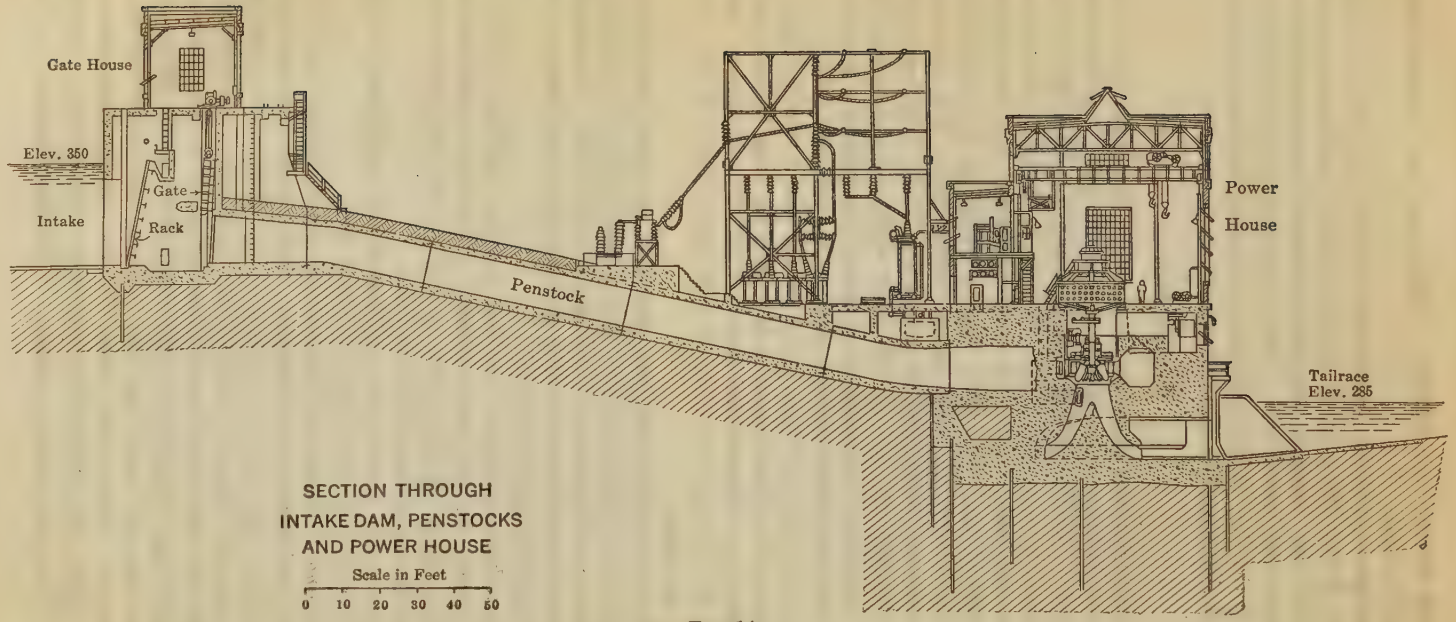

Gate House
Elev. 350
Gate
Intake
Rack
Penstock
Power
House
Tailrace
Elev. 285
SECTION THROUGH
INTAKE DAM, PENSTOCKS
AND POWER HOUSE
Scale in Feet
0 10 20 30 40 50

FIG. 14.

from the center on the forebay side; when the canal is full and the gates are closed, it is still on the forebay side of the center. Under operating conditions, the resultant is practically at the center. As the survey plugs were near the down-stream edge of the top of the dam, any slight tilting of the structure would appear in the observations as horizontal movement.

TABLE 3.—MOVEMENT OF THE INTAKE DAM.

Date.	Progressive Settlement, in Inches.						
	Survey plugs.						Remarks.
	2.	3.	5.	6.	7.	8.	
June 25, 1923.....	0.096	0.096	0.156	0.132	0.060	0.120	Canal first filled. Part of sand on penstocks in place.
July 14, 1923.....	+0.024	+0.024	+0.072	+0.012	0.060	+0.120	Canal dewatered. September 1, 1923, canal refilled and sand on penstocks completed.
October 9, 1923..	0.036	0.180	0.168	0.156	0.060	+0.060	Canal full.
May 3, 1924.....	0.036	+0.036	0.120	+0.108	0.012	0.084	Canal full.
June 20, 1924.....	+0.156	+0.108	+0.048	+0.036	+0.048	0.024	Canal full.
.....	+0.012	0.108	0.324	0.132	0.144	0.048	Net settlement.
	Lateral Movement, in Inches.						
June 25, 1923.....	N 0.108	N 0.072	N 0.108	N 0.132	N 0.036	S 0.120	Canal first filled. Part of sand on penstocks in place.
July 14, 1923.....	S 0.204	S 0.132	S 0.204	S 0.168	S 0.132	S 0.048	Canal dewatered.
September 1, 1923	N 0.060	N 0.072	N 0.096	N 0.060	N 0.072	0.000	Canal refilled and sand on penstocks completed.
April 29, 1924....	S 0.144	S 0.024	S 0.132	N 0.012*	S 0.204	N 0.012	Canal full.
June 20, 1924.....	N 0.048	0.000	N 0.036	N 0.012	S 0.024	S 0.017	Canal full.
.....	S 0.132	S 0.012	S 0.096	.....	S 0.252	S 0.173	Net change in alignment.

NOTE.—The plus (+) signs indicate a rise in the plugs due to the change in conditions mentioned under "Remarks." "N" indicates a movement north toward the forebay, and "S" a movement south toward the power-house.

* Plug probably moved.

PENSTOCKS

Water is conveyed from the intake dam to the power-house through fifteen reinforced concrete penstocks. Each wheel unit is supplied by three penstocks, 10 ft. 8 in. wide by 10 ft. 0 in. high (Fig. 15), which unite into one conduit as they enter the scroll for the turbine wheel.

Reinforcing rods, 1 in. in diameter, and bent to fit around the corners, were set about 8 in. apart. Experience showed that concreting would have been easier had the vertical side-rods been made in two parts with ends lapped at about mid-height. With the continuous rods bent over the top, it was somewhat difficult to pour and tamp the concrete on account of the close spacing of the rods.

The reinforced concrete base was laid directly on the sand, after the excavation, made by a steam shovel, had been graded by hand. A single row of interlocking steel sheet-piles was driven along each side, and also across the base above the highest visible ground-water. These piles were about 25 ft. long, and were intended to cut off the ground-water flowing under the pen-

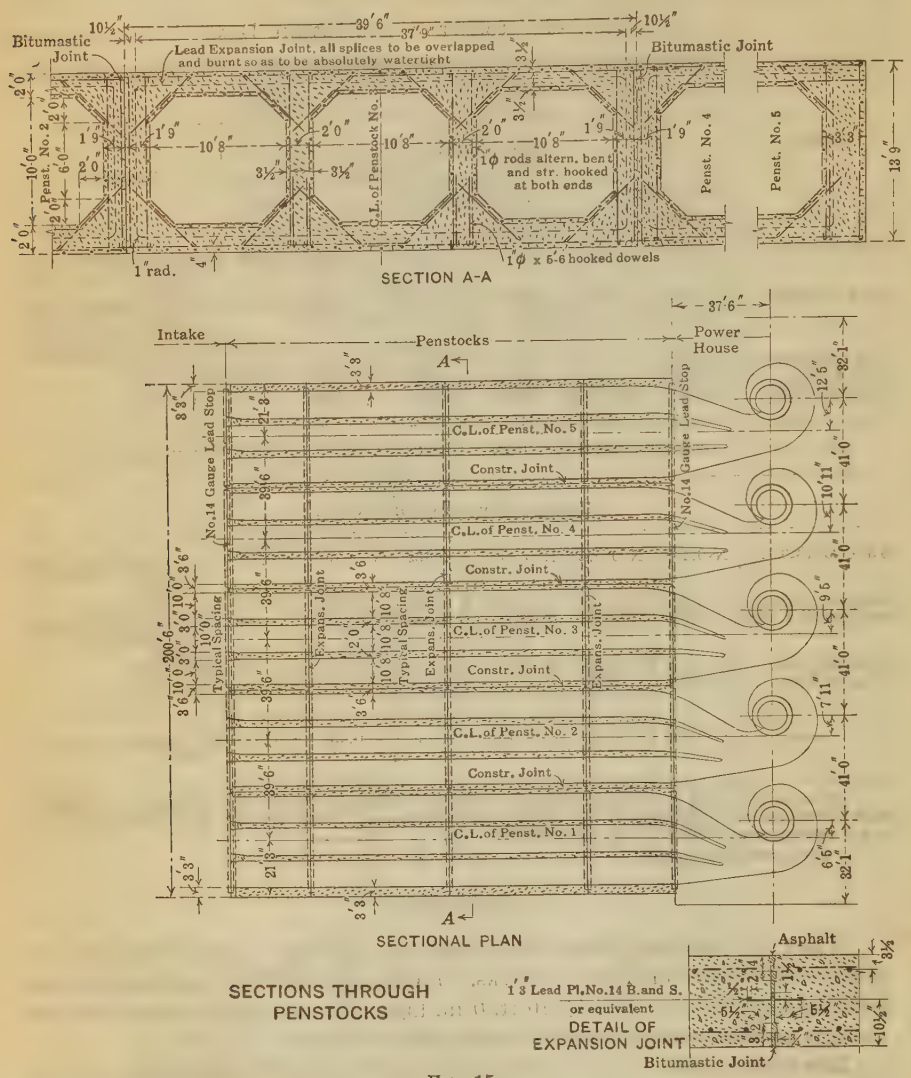


FIG. 15.

stock structure. The pouring of concrete (Fig. 13) was commenced at the power-house end, so as to work uphill. Wooden forms for the tube work were made in collapsible sections, and were kept in place for more than a week before removal.

The penstock structure was not tied into the power-house base or into the intake dam, but all construction joints had a $\frac{1}{16}$-in. lead sheet built in as a water-stop and the joints caulked with oakum from the inside. The interior surfaces of all conduits were coated with "Ferritex" after the concreting was finished.

Access into each set of three conduits is provided by a ladder in a manhole, 2 by 2 ft., constructed in a dividing wall of the middle conduit at the intake end, with passageway connections to each side conduit. The manhole and passageways also act as an air relief. There is a manhole with ladder rungs in the power-house, extending from the operating floor to a chamber having a door into the scroll of the turbines. In order to drain the penstocks when the intake gates are closed, each unit is fitted with a 10-in. cast-iron pipe (with a valve) leading from the scroll case to the draft-tube, so that the conduits and turbines can be inspected and repaired.

The lower part of the penstock structure supports concrete platforms for the outdoor transformers and lightning arresters. The upper part is covered with sand 2 to 3 ft. in thickness to reduce the temperature range as much as possible.

The penstock structure is 176 ft. in length and has five cross-construction joints, one at the intake end, one at the power-house wall, and three intermediate. When the tubes were filled with water, then emptied, and refilled, a slight vertical movement was observed at the cross-joint about 35 ft. from the power-house. After a few months' use, this fluctuation seemed to cease. When the pressure on the power-house substructure from water in the penstocks was first applied, there was an initial movement, causing leakage at the joint between the power-house and the penstocks. In order to reduce this leakage to a minimum, crimped, 37-oz., copper strips, 36 in. wide by 14 ft. long, with edges brazed, were fastened across the joint. These copper strips were held between a pure rubber gasket, $\frac{1}{2}$ in. thick by $2\frac{1}{2}$ in. wide, next to the concrete, and a wooden strip, 1 in. thick by 6 in. wide, on top of the copper, by $\frac{5}{8}$-in. brass expansion bolts and 3-in. washers. The crimping allowed for an expansion of 2 in. In each tube there was a $2\frac{1}{2}$-in. flap relief valve, brazed to the copper, opening inward, to relieve the external water pressure and prevent the copper from tearing, should the tube be empty. This remedy was successful in bringing the leakage under control. Continuous movements of the joint, as measured by a wedge and a micrometer after the initial movement occurred, indicate that the opening of the joint varies with changes in temperature.

Power-House

The power-house is 228 ft. long by 69 ft. wide, outside dimensions. Water passages and working galleries are built in the reinforced mass concrete of the substructure. The superstructure is of brick, trimmed with belt courses of precast concrete blocks. The steel roof trusses, gallery floor, and traveling crane are supported on steel columns. All the windows and skylights are of wired glass set in steel frames. The main floor was finished with a 1-in. surfacing after the electrical machinery was in place; its area is 15 742 sq. ft., or 1 sq. ft.

for each 3.2 rated horse-power of the turbines. The traveling crane, electrically operated, has a lifting capacity of 50 tons and a span of 50 ft. Heating is accomplished by two steam coils, equipped with blowers, discharging through ducts opening into the main generator room; but operation shows additional heat to be seldom required, as that given off by the generators is sufficient to warm the building in ordinary winter weather. The total weight on the foundation of the power-house, including its substructure, superstructure, and equipment, averages 3 tons per sq. ft.

The site was in shallow water near the north shore of Little Bay, a point of land protecting the house from river floods and floating ice. An earth dam was built enclosing the site, which was then de-watered. After the house and tail-race had been finished, this dam was removed by steam shovels and draglines.

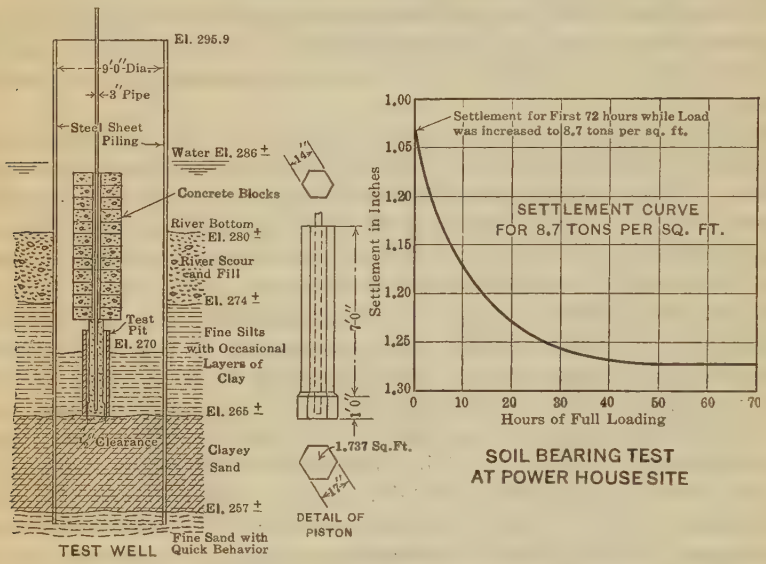

El. 295.9
9'-0" Dia.
3" Pipe
Steel Sheet Piling
Water El. 286 ±
Concrete Blocks
River Bottom El. 280 ±
River Scour Sand Fill
El. 274 ±
Test Pit El. 270
Fine Silts with Occasional Layers of Clay
El. 265 ±
Clearance
Clayey Sand
El. 257 ±
Fine Sand with Quick Behavior
TEST WELL
1.737 Sq.Ft.
DETAIL OF PISTON
Settlement in Inches
Settlement for First 72 hours while Load was increased to 8.7 tons per sq. ft.
SETTLEMENT CURVE FOR 8.7 TONS PER SQ. FT.
Hours of Full Loading
SOIL BEARING TEST AT POWER HOUSE SITE

Fig. 16.

The sand deposit at the power-house site was of such nature that great care was required during construction. After information collected by test borings was studied, a well was dug inside a steel sheet-pile casing, 9 ft. in diameter (Fig. 16). A dark fine sand layer containing some clayey characteristics was reached at about Elevation 265. This was the blanket or impervious layer previously mentioned, which overlaid the fine sand with "quick behavior" that had an Artesian pressure due to ground-water. A test pit was then arranged in the well; the piston, having an area of 1.737 sq. ft., was loaded with concrete blocks, and the settlement measured (Fig. 16). Under a weight of 8.7 tons per sq. ft., after 50 hours, no further settlement of the piston occurred. This loading was 2.9 times the unit weight of the completed building.

As a search by test borings and surface observations did not indicate any better location, the power-house excavation was commenced at the site selected. Two rows of steel interlocking sheet-piling were first driven, one row under the outer edge of the substructure, and a second about 5 ft. inside and parallel to the first. The outer row was driven about 13 ft., and the inner row about 30 ft., below the bottom elevation of the concrete base. Then cross-rows were driven in both directions, dividing the area into ten rectangles about 41 by 35 ft. in size.

When the work began, it was evident that in the wet excavation great difficulty would be experienced in holding the bank on the north side across a wide excavation, without an equivalent burden of sand on the south side. The flat slope of repose of the fine wet sand in the water-bearing strata of the north bank made it expedient to disturb the bank as little as possible. These conditions dictated the next step, which was to drive a third row of sheet-piles along the north side next to the bank, about 21 ft. inside the outer row, and to excavate sections between these rows. When the desired sub-grade was reached, the sections were filled with concrete to Elevation 284. This concrete served temporarily as a retaining wall against the sand bank, while the excavation for the remainder of the substructure was being made.

Then the spaces between the two outer rows of sheet-piling around the other three sides of the power-house site were excavated to grade and filled with concrete, following which, the interior rectangles were excavated by a clam-shell bucket, one after another, until there was risk of creating boils of dangerous size through which the fine sand from below might flow. This risk was indicated by the beginning of small boils in isolated localities. When these small boils gave evidence of increasing, the holes were filled with sand and broken stone. This expedient was successful; and the excavation was carried to the desired elevation, namely, about 15 ft. below the original sand bottom, the water being kept under control by pumping from sumps in neighboring rectangles. The sumps were filled with broken stone which acted like filters and minimized the danger of pumping out sand. While the sump pumps kept the water under control, a sub-base of concrete, 12 in. thick, was deposited in the rectangles by self-dumping buckets. This prevented further boils. On top of the sub-base, the substructure concrete was poured in alternate rectangles, to keep the loading on the sand as uniform as possible over the area. All irregularities in settlement were corrected by bringing the concrete in the various rectangles to uniform levels as the work progressed. Cross-braces between the rows of sheet-piling were removed and the sheet-piles burned off at the time the concrete was carried up to the level of the bottom of the draft-tubes.

The settlement of the structure was measured as the work progressed. The greatest settlement, caused partly by pumping, was $2\frac{17}{32}$ in., near the southeast corner, of which 2 in. occurred in the sub-base before the substructure concrete was poured. The least settlement was $1\frac{3}{32}$ in. and the whole averaged about $1\frac{15}{32}$ in., being fairly uniform, except at the one corner. Subsequent surveys showed $\frac{1}{8}$ in. additional settlement, when the superstructure

was erected, after which no further settlement was noted. As previously mentioned, when the penstocks were first filled, the water pressure on the power-house caused a slight horizontal movement.

The draft-tubes are annular, of the Moody spreading type, with a central concrete cone, the armored apex of which is 5 ft. below the bottom of the runner. The discharge into the river from each wheel is through two openings, each 15 by 11 ft. (Fig. 14).

Considerable vibration occurred as each unit operated through a range between 3 500 kw. and 4 500 kw. at ordinary tail-water level. This vibration was traced to the formation of a vacuum between the apex of the concrete cone and the bottom of the runner. The trouble was remedied by inserting a 4-in. pipe through a stuffing-box fastened to the manhole cover of the inspection gallery. The pipe extended about 18 in. into the draft-tube, and was fitted with a coiled spring foot-valve at the outer end in the gallery, and a vacuum gauge and stop-valve at about mid-length. By operating a unit through the full range of gate opening and by manipulating the stop-valve while observing the vacuum gauge, the foot-valve was adjusted to open and close automatically within the desired range of vacuum. This simple device, suggested by C. A. Terry, Superintendent at the power-house, worked well, and effectively stopped the vibration and water-hammer.

Tail-Race.—Water discharged from the turbines passes directly into the river. The elevation of the bottom of the discharge openings is about 9 ft. below the natural bed of the river, and, therefore, the area was excavated to grade. As the discharge channel widened, the depth of cutting was reduced until the natural level of the river bed was again reached. A concrete pavement to prevent scouring (Fig. 14), was laid along the discharge front of the power-house, extending out 50 ft., and inclining upward on the gentle slope of the excavation. For the first 20 ft., this slab is 30 in. thick and is reinforced between the buttresses of the turbine discharge passages, which buttresses extend out on the slab to resist any uplift pressure. For the remaining 30 ft., the slab is thinner and unreinforced.

Log Chute

As the river bed below the main dam is practically dry when water is not passing over the spillway, a logway was built in the head-works to float logs, whenever necessary, into the canal. A chute was built in the intake dam to pass these logs and excessive ice accumulation into the river.

This chute consists of openings in the intake dam, each 8 ft. wide by 9 ft. high, closed by concrete gates, which connect with two passages through the dam. The two passages unite at the discharge portal into a timber log chute, which extends to the river at the east end of the power-house.

Closure of River

As a start in construction operations on the main dam Lackawanna steel sheet-piles, to form the first coffer-dam, were driven from the north bank to the center line of the bay between Buttresses 10 and 11. These piles were about 35 ft. long, with tops at Elevation 317, and penetrated the sand about

18 ft. The rows were braced against log cribs, 10 by 30 ft., filled with stones. The interior was de-watered by motor-driven centrifugal pumps. The top layer of loose material was removed, and trenches were dug for the cut-off ribs. The concrete base was then laid, and the buttress forms were set up and poured, beginning at the north end.

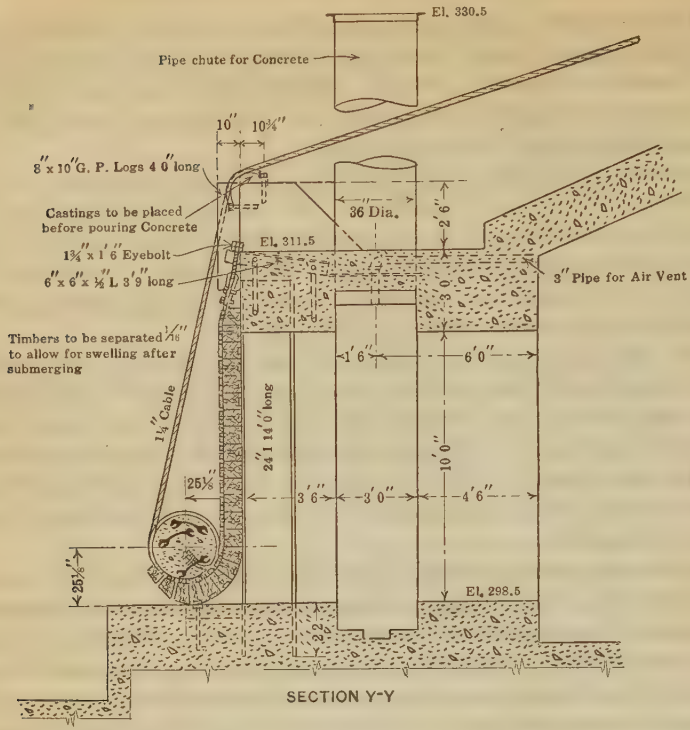
A cross-row of piles was driven on the center line of Buttress 12, joining the up-stream and down-stream rows. The tops of these cross-piles under the dam were cut off to end in the concrete base of the dam, while those on both the up-stream and down-stream sides of the structure projected to Elevation 317. This cross-row formed the north end of the second coffer-dam for constructing the south portion of the dam.

In the beginning, the river flowed through the channel left between the end of the first coffer-dam and the south bank. After the north portion of the dam was built, the up-stream coffer-dam piling was burned off at about Elevation 302, during low water, and the down-stream piling was pulled. The river then was allowed to flow through the bays between the buttresses in the north portion, while the second coffer-dam and south portion of the main dam were being built.

When the dam was completed across the river, the up-stream and down-stream sheet-piling of the second coffer-dam were burned off, the down-stream row at about Elevation 301. Stop-logs were placed in grooves at all the bay openings except four, namely, between Buttresses 5 and 6, 6 and 7, 8 and 9, and 9 and 10, and mass concrete was poured between the stop-logs and bulkhead forms at the upper end of the bays. The river then flowed through the four bays left open, and in case of emergency there were the eleven openings in the spillway dam. Of these latter openings, four were temporary, each 9 ft. wide, and seven were permanent, each 5 ft. wide, closed by wooden sluice-gates.

When the time arrived finally to close the last four bay openings, roller-gates (Fig. 17) were constructed, similar in principle to those used by M. Caméré in France. The rollers were steel pipes, 36 in. in diameter, filled with concrete and old iron, so that they would sink by their weight. A curtain of 7½ by 7½-in., dressed yellow-pine timbers, hinged together by 1-in. plow steel rope, was wrapped around the central pipe core, with one end fastened to the dam at the top of the opening and the other end to the core. Cables of 1¼-in. wire rope were wrapped around the projecting ends of the core, and carried to a windlass on the dam. A heavy canvas covering was placed over the timbers on the up-stream side to make the dressed joints as tight as possible. Adjacent sides of the logs were slightly beveled, so that their faces would fit tightly together. The roller-gates were lowered and raised a number of times before their final closing.

When all was in readiness, the four roller-gates were lowered in quick succession, and concrete was placed behind them in the bays, by chuting it through 36-in. steel pipes, the upper ends of which were above the temporary pond level. The river then passed entirely through the openings in the spillway dam into the waste channel.



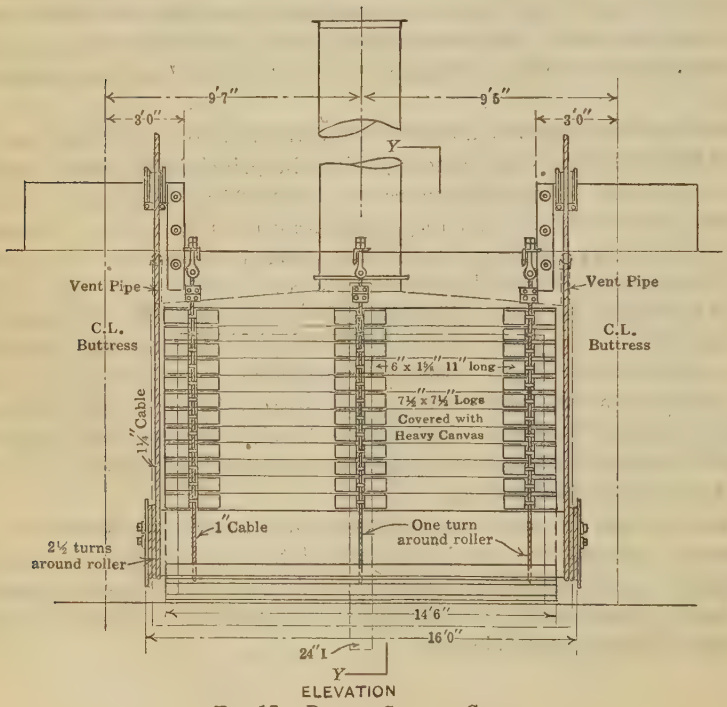


FIG. 17.—ROLLER CURTAIN GATE.

The temporary spillway openings were stopped by logs and filled with concrete, leaving the river to pass through the seven permanent gate-openings. These gates were closed as the final act, and the water rose in the pond until it passed over the crest of the spillway.

Pressure Test Pipes

The original design contemplated thirty-nine test pipes, three in each alternate bay, evenly spaced up stream and down stream. The pipes were 3 in. in diameter, of extra heavy wrought iron, shod with a point, and perforated with 160 holes, $\frac{1}{8}$ in. in diameter, for a distance of 10 ft. just

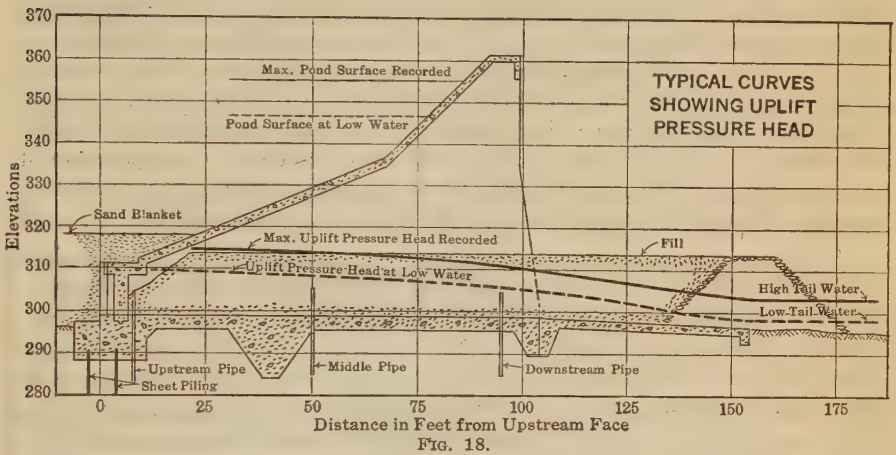


FIG. 18.

above the point. These pipes extended under the base slab, about 10 ft. into the sand. Each pipe was to have a composition cap, tapped for a $\frac{1}{2}$-in. extension. As constructed, seventeen pipes were placed, with four bays having two pipes and three bays, three pipes each, the up-stream and down-stream spacing being irregular.

The uplift pressure was measured by the height of the water standing in these pipes. It was found that the pressures were not the same in all the bays, but, in general, were greater under the northerly portion (where the sand in the river bed was finer) than under the southerly portion.

Although the uplift pressure is a function of the total head, as measured between water elevations above and below the dam, its magnitude varies with temperature and other conditions. The elevation of the water surface in the pipes varies closely with changes in elevation of the river surface just below the dam, while changes in pond elevation have less relative effect.

Fig. 18 shows typical hydraulic grade lines through the dam. The average loss of head due to the sheet-pile cut-off is about 38 ft. The distance from the top of the sand fill above the dam to the bottom of the sheet-piling is about 80 ft., and the distance from the bottom of the piling to the bottom of the dam is about 50 ft., or a total of 130 ft. The average loss of head in passing through the sand and boulder foundation under the dam and apron is about 11 ft. in a horizontal distance of 150 ft.

The average uplift pressure head (the average of all readings taken on the same day of the difference in water level in the pipes and tail-water) reached 11.3 ft. when the net head between the pond and tail-water levels was 50.5 ft., or about 22% of the net head created by the dam. The original calculations included an allowance of 26 per cent. Although the design considered the uplift pressure as acting on the entire base, the actual uplift pressure probably acts on less than 50% of the area, due to sand contact. The head as measured in the test pipes is practically the total head, as the velocity head would be negligible. The sheet-piling proved extremely efficient as a cut-off, considering the difficulty experienced in driving it through the boulder layer. It would be well for other engineers to record their experiences with steel sheet-piling used as a cut-off under actual working conditions.

In Fig. 19 is illustrated an interesting relation between the seasonal water temperature and the average uplift pressure. Curves platted for individual pipes show the same characteristics as the average given in Fig. 19. The writer believes that this is the first record of the effect of temperature of water on uplift, as registered under actual conditions, for a large structure on sand. It will be noted that the difference in uplift between low and

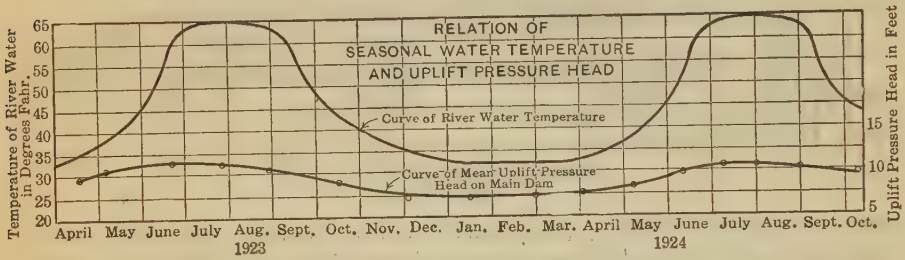


FIG. 19.

high temperature periods amounts to about 65% of the minimum uplift. Under other structures of similar character, it is possible that observed uplift pressures might cause anxiety unless the effect of temperature was considered.

The water temperature varies from 65° Fahr. to about 32° Fahr., or through a range of 33° Fahr. (18.3° cent.). Meanwhile the average uplift head varied from 11.3 ft. to 7.2 ft., or, say, 1 ft. for each 8° Fahr. (or 4½° cent.) change in temperature. There was no way of determining the actual temperature of the water in the sand under the dam, but it probably differed little from that of the river water, although the range in temperature may have been somewhat smaller. The character of the measurements was such as not to permit of more than general conclusions, namely, (1) that the velocity of water under the dam was less than 0.001 ft. per sec. at the maximum temperature recorded, as based on Hazen's formula,* using a constant of 800 and an average effective size of sand of 0.5 mm.; and (2) that an increase in the temperature of the water, having the effect of decreasing the resistance to flow through the sand, not only increases the

* *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 201.

velocity, but also decreases the loss of head. The increase in velocity from the lower to the higher temperature was probably less than 60 per cent.

It will be noticed that the maximum point of the uplift curve for 1924 is less than that for 1923, although the maximum water temperatures were the same. This may be accounted for by a silting up of the pond and by a compacting of the sand blanket at the heel of the dam.

Concrete

For various parts of the work, the mix by weight was as follows, the required compressive strengths being for specimens 28 days old:

For arches, struts, and flood-gates (2 400 lb. concrete): 1 part cement to 6 parts aggregate (equivalent to 1:2:4); and for each 100 lb. of cement not more than 8 gal. of water.

For other reinforced concrete, such as the bottom slab, head-works, intake, power-house (2 200 lb., concrete): 1 part cement to 6.5 parts aggregate (equivalent to 1:2.17:4.33); and for each 100 lb. of cement not more than 8½ gal. of water.

For buttresses and canal lining (2 000 lb. concrete): 1 part cement to 8 parts aggregate (equivalent to 1:3:5); and for each 100 lb. of cement not more than 9½ gal. of water.

For the spillway dam, apron, training wall, and other mass concrete not reinforced (1 600 lb. concrete): 1 part cement to 9 parts aggregate (equivalent to 1:3:6); and for each 100 lb. of cement not more than 10 gal. of water.

Lime was added as follows: 6% for arch concrete; from 6 to 8% for the penstocks; 7% for the canal lining; but none for the buttresses, base slab, and similar concrete.

The cement used was Ironclad, made by the Glens Falls Cement Company, and shipped in bulk by railway to the site. Sand was selected from the better grades found in the excavations. A crusher plant furnished the broken stone, using selected rock taken from the excavations. Considerable trouble was experienced due to the hardness of the rock which damaged the crushers from time to time.

The main concrete mixing plant was equipped with a mechanical weighing device for controlling the mix. This device, designed by Nathan C. Johnson, Assoc. M. Am. Soc. C. E., consisted of a scale carrying the charging hopper on one side and adjustable weights on the other, which automatically shut the gates of the bins when the proper weight of cement and sand, or stone, had been received in the hopper.

Stone from the crusher contained a substantial proportion of "fines" and dust which would pass a No. 4 screen. The maximum size of crushed stone was 2½ in. Sand from the excavations contained a varying quantity of gravel which would not pass through a No. 4 screen, but all gravel larger than 1½ in. was removed by a "grizzly." Diagrams were prepared showing the proper proportion of sand and stone to be weighed into the mix for all possible variations of "fines" in the stone and of gravel in the sand, so as to obtain the strength required. Notwithstanding the care taken by the Chief Inspector, William A. Aiken, M. Am. Soc. C. E., in changing the

proportion as indicated by mechanical analyses, it was found that uniform strength, with the same quantity of cement, could not be obtained until the "fines" were removed by the installation of a "dust jacket."

Bin cement was tested at the Glens Falls Laboratories of the International Paper Company, while cylindrical test pieces, cast in special pipe forms, 10 in. in diameter by 16 in. long, were sent regularly to the testing laboratories of the Rensselaer Polytechnic Institute at Troy, N. Y. As an added precaution, test pieces were frequently cut from the finished work and sent to the laboratories.

For concrete mixed in freezing weather the aggregate was heated, and after depositing, the temperature was kept at about 50° Fahr., or more, for at least 72 hours, by using salamanders and by covering the surfaces. All important forms were coated with oil and wetted before pouring the concrete. Forms were kept in place for a minimum of five days, but when the daily temperature fell below 70° Fahr., the minimum time was lengthened.

Quantities

The following quantities were used in the finished structures, exclusive of structural steel and cement for grouting and "guniting":

Rock excavation	159 899	cu. yd.
Sand and earth excavation	799 160	" "
Trimming canal slopes	68 600	sq. yd.
Sand and rock-fill	118 352	cu. yd.
Rip-rap	11 870	" "
Concrete	127 803	" "
Pre-cast concrete trim	4 849	lin. ft.
Reinforcing steel	1 973	tons
Expanded metal	291	"
Steel sheet-piling	2 335	"
Brick	826 790	
Expansion joints	43 405	lin. ft.

In addition, considerable quantities of excavation, fill, lumber, concrete, and other materials were used for the falsework, camp, railroad, river cribs, and the like.

Equipment

The principal items of equipment were as follows:

One 50-ton Shaw over-head traveling crane, electrically operated.

Four turbines, William Cramp and Sons, I. P. Morris Department; head, 66 ft.; speed, 150 rev. per min.; 10 000 b.h.p.; vertical shaft; Kingsbury bearing (fifth turbine to be installed later).

Four main alternating generators (fifth generator to be installed later), with field switch panels; General Electric Type A. T. B., Form V; 48 poles; 9 000 maximum k-va.; 150 rev. per min.; P. F. 8; 7 200 kw.; at full load, 6 600 volts.

Four exciters, mounted on top of the alternators; General Electric direct-connected, Type M. C. P., Form A; 8 poles; 114 kw., maxi-

mum; 150 rev. per min.; 456 amperes; at full load, 250 volts; continuous 50° cent. rise.

One motor generator set, as extra exciter; induction motor, 1 E. 13 B, Form K; 3-phase; 60 cycles; 550 volts; 153 amperes; 1 170 rev. per min.; direct-connected to direct current generator, shunt wound, Type R. C. 37; 250 volts; 460 amperes; 1 170 to 1 200 rev. per min.

Four hydraulic governors, with Taylor Control System, William Cramp and Sons, I. P. Morris Department, Serial No. 152.

Three Woodward oil pumps to governors, link belt driven by induction motors, Type K T.; 3-phase; 40-cycle; 550 volts; 30 amperes; 765 rev. per min.

One air compressor for five accumulators.

One station switch-board; with control bench board and voltage regulators.

Seven subtractive polarity transformers; General Electric Type F. O., Form HDH.; volts, 6 600 to 114 500; cycles, 60; capacity, 7 500 k-va.; continuous 55° cent. rise; two banks of three; one spare.

Five high-tension oil circuit breakers; General Electric Type O. F. 1, Form B. 1, Class F. 2; maximum amperes, 400.

Three multi-whirl coolers, Griscom Russell Company; size, 111-60; shell, 50 in.; tubes, 50, driven by induction motors; General Electric Type K T., Form C; volts, 550; amperes, 14.7; speed, 1 760 rev. per min.

Eight oxide film arresters; alternating-current; General Electric Type O. F., Form H. O.; volts, 92 400 to 115 500.

Transmission line cables; 7 wires, No. 0000; 114 000 volts.

Three lighting transformers; Type H, Form N.

Three station transformers; Type H; 100 k-va.; cycles, 60; volts, 6 900 to 11 950.

One set of storage batteries, with charging motor-generator set.

Twelve gate-hoisting gears (three more to be installed) for Broome gates at intake dam; Maine Electric Company; driven by induction motors, Type K T., Form B; 3-phase; 60-cycle; 15 h.p.; 550 volts; 17.5 amperes.

Construction Plant

A standard-gauge, single-track railway, with sidings, was built over a leased right of way from Glens Falls to the site, a distance of about 7½ miles, with branch tracks to reach all parts of the development. Tracks were extended over bridges, both just above and just below the main dam, so that easy communication was had with the south bank. The bridge girders rested on the cribs supporting the steel sheet-piling of the coffer-dams.

Derricks were erected and shifted to serve all parts as the work progressed. A Lidgerwood cableway, steam-operated, was stretched over the center line of the main dam. The span was about 1 550 ft., the cable, 2½ in. in diameter, with a maximum allowable center load of 10 tons.

The stone crushers, cement storage, and principal mixer plant were on the north bank near the end of the main dam. Smaller mixers were located at about mid-length of the canal and at the forebay dam. Concrete was distributed by chutes radiating from towers, and also by dump-cars to those parts which could not be reached by the chutes.

A camp was built on the north bank, with completely equipped bunk-houses, dining room, cafeteria, kitchen, bakery, store, first aid hospital, machine

shop, blacksmith shop, carpenter shop, etc. There was also an office building, with accounting and drafting-rooms for the engineers and contractor's staff, and a number of houses for the contractor's officers and their families, some of which were designed for permanent use by the Operating Staff.

A reservoir, for drinking water supply, was built on a neighboring hill, while an elevated water tank, supplied by a pump from the river, served for fire protection, construction, and sanitation. There was also a drainage system throughout the camp.

During active construction, the working force varied from 1 000 to 1 500 persons. Some employees resided in Glens Falls and were brought to and from the site by train, with free transportation.

Electric current for light and power during construction was supplied over a temporary transmission line from the Glens Falls Mill of the International Paper Company.

Personnel

A. H. White, M. Am. Soc. C. E., Chief Engineer of the International Paper Company, had general charge of the work, assisted by Edward Hutchins, Assoc. M. Am. Soc. C. E., Assistant Engineer; Chester S. Colson, Hydraulic Engineer; W. H. Coventry, Electrical Engineer; H. M. Hale, M. Am. Soc. C. E., Resident Engineer; and A. H. Davison, Engineer Observer and Recorder. The writer was Consulting Engineer, and was assisted by Mr. Vincent J. McKinnon and George Perrine, M. Am. Soc. C. E. The work was carried out under a modified cost-plus contract and detail drawings were furnished by the Parklap Construction Corporation, of which Walter J. Douglas, M. Am. Soc. C. E., was President; Eugene E. Halmos, M. Am. Soc. C. E., Chief Engineer; S. A. Thoresen, Designing Engineer; E. A. Little, Resident Manager; G. Miller, M. Am. Soc. C. E., Field Engineer, and F. W. Barnes, Assoc. M. Am. Soc. C. E., Superintendent. J. P. Hogan, M. Am. Soc. C. E., gave advice at various times. The entire personnel of the International Paper Company and of the Parklap Construction Corporation deserves credit for faithful application under the many difficult conditions that arose during the progress of the work.

DISCUSSION

EUGENE E. HALMOS,* M. AM. SOC. C. E.—In his paper the author has presented a complete description of the structures of the Sherman Island Development and several articles in engineering periodicals have contained elaborate accounts of the methods of construction. There is little, therefore, to be added. This discussion will be confined to the design of the coffer-dams in which the main dam was built and will show its correlation with the general program of construction and with the flood characteristics of the Hudson River.

The magnitude of the project indicated a period of construction extending over approximately two and one-half years, allowing two seasons for the construction of the main dam. It was decided to construct the dam in two sections beginning with the northern part. As soon as the spring freshets of 1921 receded, the first coffer-dam was to be built adjoining the north shore, leaving the southern unobstructed part of the river bed to carry the flow. The completion of the dam in this first coffer to Elevation 337, was required by February, 1922. Immediately thereafter, and before the expected occurrence of the 1922 spring floods, the sheet-piles of the first coffer were to be pulled and later re-driven after the 1922 freshets had receded, to form the second, southerly, coffer-dam. During this second season of building operations the dam in the second coffer was to be constructed and the part built in the first coffer topped off, while the flow of the river was to pass through the openings provided for this purpose between the buttresses of the first part of the dam.

The basis of the design, therefore, consisted in fixing the length and height of the first coffer-dam so as (1) to leave an unobstructed waterway to discharge the maximum summer flood; and (2) to construct a sufficient number of bays within this dam for the discharge of a similar summer flood after the second coffer-dam had been built to the same height, thus closing the waterway.

The calculations were based on the available data, consisting of fairly complete flow records, a rating curve for the dam site, and a cross-section of the river. The orifice formula was used to determine the discharge capacity of the sluices in the dam, the coefficients being taken from the results of measurements on the sluices of the Assuan Dam. The submerged weir formula was applied to the open sections and the effect of back-water was given proper consideration, following a trial-and-error method, until a perfect balance was obtained. The crest of the coffer-dam was set at Elevation 317.5, which is about 22 ft. above the general level of the river bed. In the first coffer, a length of about 350 ft., was necessary to permit the construction of 16 bays giving the required carrying capacity for the discharge of 36 000 cu. ft. per sec., the maximum summer flood on record, with 6 in. of free-board at the up-stream line of sheet-piles. The same carrying capacity for the same height of water level was provided in the 140-ft. open waterway between the south end of the first coffer and the south shore of the river. Fig. 20 shows the discharge capacity of the openings in the dam and of the open waterway, for different water elevations up stream from the coffer-dams. The sheet-piles

* Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

were 45 ft. long along the up-stream line and 35 ft. along the down-stream line, one-half the length being driven into the river bed. They gave an excellent water seal and occasioned no difficulty in keeping the coffer-dams free of water for working purposes.

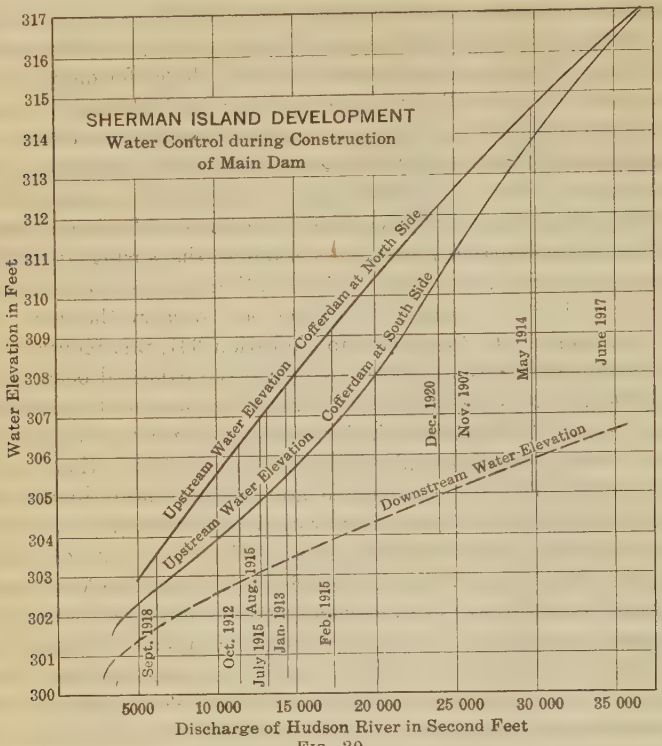


FIG. 20.

It was also estimated that with the openings in the northern part of the dam available for flood discharge plus the capacity of the open waterway, the height of the water above the dam for a flood of 100 000 sec-ft. would not have been over Elevation 330. This head was thought to be the maximum to which the dam could be safely subjected without using the up-stream and down-stream fills to lend additional weight and stability to the structure. The actual discharge of the 1922 flood reached a maximum of 56 000 sec-ft., raising the water behind the dam to Elevation 322.

The original program was carried out to the letter except that the up-stream row of piling in the coffer-dam was not pulled, but instead was cut at low-water elevation so as to provide an additional cut-off. The dam was ready for the final closing operation by the end of 1922. This final closing, however, was considered to be such a delicate operation, and involved the other structures of the development to such an extent, that plans were made for its execution as early as August, 1922. These plans were based on assumptions of expected weather and flow conditions and were flexible enough to be changed to conform

to the actual conditions prevailing at the time of closure. Fig 21 shows the discharge capacity of the openings in the main dam and in the spillway dam, which were provided to control the elevation of the pond during the operation of closing.

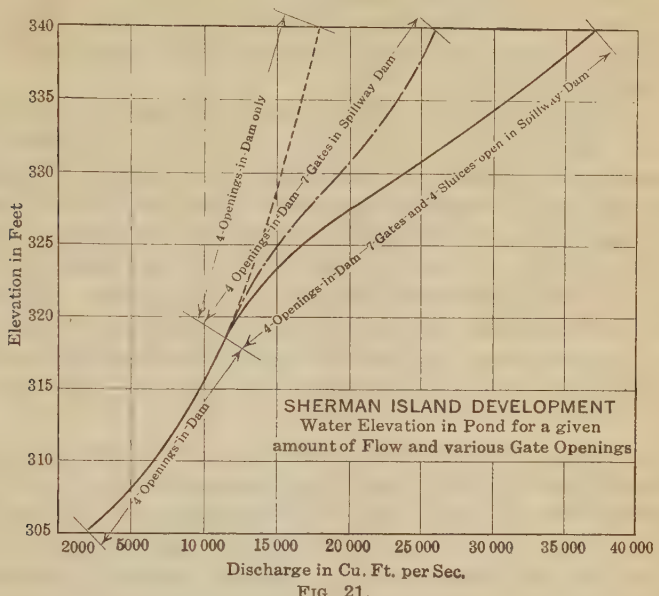


FIG. 21.

The time of the actual closing was delayed on account of the extreme cold weather in January and February, 1923; the first water went over the spillway dam on March 29, instead of March 1, 1923, as originally contemplated.

J. C. MEEM,* M. AM. SOC. C. E.—The speaker has been greatly interested in this paper and more especially in that part of it relating to uplift.

It appears that the author has given a large measure of concise, definite, and practical information on this interesting subject. For some years the speaker has been interested in this question, often misunderstood, and has made, in a small way, some experiments and investigations concerning it and its allied subject—water pressure through soil. It is believed that the following may be pertinent, even though some of it has appeared before in the *Transactions* of the Society.

In a paper† read before the Society on May 18, 1910, the speaker described an apparatus and method for measuring the uplift through soil, which, briefly described, consisted of a hydraulic chamber connected by copper tubing with a pump and gauge. Projecting into this chamber, through hydraulic packing, was a piston 2½ in. in diameter. With the piston plunged about 6 in. into water, partly filling the chamber, it was found that a gauge pressure of approxi-

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† "Pressure, Resistance, and Stability of Earth," *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 352.

mately 8½ lb. was required to overcome its weight and the small frictional resistance.

With the piston plunged into sand carrying an additional head of water of 2 or 3 in., it was found that after it had stood for about 2 hours, it required approximately two and one-half times the pressure, or nearly 22 lb. (average), to start it, after which the pressure fell to 8½ lb. as before. This apparently indicated that the pressure area was only 40% of the whole piston area until the sand contact was broken.

The speaker does not claim that this experiment was on a sufficiently large scale to be definitely conclusive in its results, but he believes it shows unquestionably the principle of diminished area of pressure through soil.

It was especially interesting, therefore, to note the author's findings, not only as to the varying loss of head due to temperature changes—loss of head due to the sheet-pile cut-off—but especially as to the general loss of pressure due to the diminished area, which he estimates at 50% (although in the interest of safety making allowance for the full amount). In all structures exposed to water pressure and resting on soil, the speaker estimates that the theoretical uplift will not exceed 40%; but, for safety, he uses 50% as a factor.

Because a gauge, or a number of gauges, through the walls or roof of a structure shows full water pressure, many engineers have assumed that there is full water pressure over the whole area. Water pressure will find its way to any orifice and, if not relieved, nor too distant, nor transmitted by too tortuous a way, will show full pressure on any gauge or number of gauges, although the intervening areas of soil contact may be completely free from pressure.

The speaker has recently noted in the pulling of sheet-piling driven to a penetration of more than 30 ft. below the bottom in tidal water and into a dense soil containing a mixture of blue clay and fine sand, that, in many instances, the piling came up with large lumps of soil adhering to it, some of the masses being in excess of 1 ft. in thickness, with their interiors more or less dry. This indicates that water which does not have an outlet, is excluded from soils somewhat in proportion to their densities.

The soil in question is similar in texture to that underlying the Hudson River and is often classed as quicksand, because of its tendency to become "soupy" under hydraulic action.

The fact that structures resting wholly on soil do not have water pressure over their full area should not be confused with the pressure through soils against structures resting on piles or caissons, which may be subject to full pressure over all areas not in contact with the piles or caissons. This was especially noted in the case of the dry dock at the Brooklyn Navy Yard, which is largely supported by wall caissons.

The value of the pile cut-off is clearly demonstrated in this paper; but apparently there should be no question as to its necessity. The impounded water must find an outlet to and through which seepage may occur. Without the pile cut-off, this seepage might develop slow erosion, eventuating more or less disastrously, whereas, with the cut-off in place, greatly reduced seepage

continues, as shown by the pipe tests; but erosion cannot occur, and it is probable that in time the seepage may be further reduced or entirely prevented.

GEORGE A. ORROK,* M. AM. SOC. C. E.—This paper shows once again the essential tightness of hardpan to resist the percolation of water. Engineers learned this early in the building of dams. They were taught to cut a trench across the stream until they struck the hardpan, to put in a mud-sill, to drive the sheet-pile cut-off on the up-stream side of the mud-sill, and to erect A-frames and a wooden structure, which could be covered over with sand or loam on the up-stream side. Thus, although they did not know the use of concrete, they had the prototype of the concrete dam in wood.

Glacial drift hardpan is a much better material than river drift. The latter material is not hardpan by any means, being sometimes found at least 100 or 200 ft. in thickness in the rocky bottoms of the river valleys. It has been bridged over in a number of dams in New England, notably on the Shetucket River, by the Scotland Dam built a few years ago, and just below it by one probably 45 years old at the Village of Baltic, Conn.

These dams have rocky foundations on either side of the river, but none at the center, or at least within any ordinary depths. The Scotland Dam was built only a few years ago, steel sheet-piling being used for the cut-off; and there has been no passage of water under the cut-off piling, on the down-stream side, as far as can be observed. At the Baltic Dam, however, the builders followed the old plan, using wooden sheet-piling, because, in those days, steel sheet-piling was unknown. The consequence was that the water followed the cut-off in the river drift on the up-stream side of the piling until it got below it, and passed up on the down-stream side washing out the foundation, resulting in a large hole on one side of the dam and the emptying of the pond.

This was corrected by using a sheet-piling that went down 7 ft. farther, and an attempt at grouting, which, in those days, was not a very great success. Seventeen years ago that second piling went out, but not in the same place as the first. Then the engineers did what they should have done in the first place—they placed a proper cut-off along the whole length of the dam; they went down to about 45 ft. To date, that cut-off has given good service.

LAZARUS WHITE,† M. AM. SOC. C. E.—The Engineering Profession is indebted to Mr. Parsons for the experiments and investigations made in connection with building a dam on sand foundations, and for his clear exposition of the problems involved; also, of course, to those who had the courage and faith to finance the construction.

Of particular interest are the observations and data bearing on uplift pressures. The subject of uplift beneath dams has been discussed to the speaker's knowledge for the past twenty-five years, and estimates on its value have varied from 100% of the theoretical downward. It is evident that it must be much less than 100% or a great many existing dams would not now be

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† Pres., Spencer, White and Prentis, Inc., New York, N. Y.

standing. Mr. Meem, in his discussion, refers particularly to his experiments as published in his paper entitled "Pressure, Resistance and Stability of Earth."* These experiments, which are still the only good ones, demonstrate that the upward pressure is in proportion to the voids in the sand. As he did originally (1910), Mr. Meem again calls attention to the need of further experiments on a larger scale.

It is surprising that these have been the only tests, considering their great value and the need of conclusive data on this point. For lack of precise information many structures have been extravagantly designed. It would appear that by a relatively small expenditure, this problem could be solved.

An interesting feature of the paper is the information given on the effect of steel sheet-pile cut-offs and on the variation in uplift on a dam between winter and summer. This may be explained by the viscosity of water, which property is more important than is ordinarily considered. As water becomes denser with lower temperatures, the viscosity necessarily increases, thus counteracting the hydrostatic effect of the water contained within the grains. Water flowing in small spaces acts like a glue, causing great loss of head.

The true value of uplift is of great importance in the design of coffer-dams, as, for instance, where steel sheet-piling is driven in a river bed of sand, the material dredged out, and a concrete blanket placed by tremie to allow the subsequent pumping out of the coffer-dam. What thickness of blanket is to be used? The full theoretical pressure for a 40-ft. head would require a blanket almost 20 ft. thick, whereas a blanket less than 10 ft. thick has been shown by experiments to be ample. Because of lack of precise data on this question, much concrete is wasted. The excess of concrete in a single coffer-dam would pay for the experiments necessary to clarify the whole subject.

Thaddeus Merriman,† M. Am. Soc. C. E.—It has been most instructive to study this paper because it so clearly describes a work which was done under difficult and trying conditions. Certain features are particularly worthy of discussion. Among them is the tabulation on page 1263, which discloses the surprising fact that the greatest known flood occurred in 1913—within the lifetime of a small boy. No record of the greatest unknown flood is presented. The only certainty in regard to flood records is that they are all sure to be exceeded. The largest known flood is always the one which is displaced from its high position on the list. Few, if any, flood-flow records have even the respectability of middle age. Nearly every day, in some part or other of this fair land, there occurs an unprecedented flood, leaving behind it desolation and destruction. The present tendency seems to be that of concentrating all kinds of detailed analysis on such meager data as are available, in the effort to predict what the future may bring forth. This procedure naturally results in the exclusion of both judgment and vision. The records left on the face of the topography by the great floods of the past are seldom, if ever, included in the computations on which the predictions for the future are based.

* *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 352.

† Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

The cross-section of the dam site (Fig. 3) is incomplete to the extent of not showing where the rock lies under the central part of the structure. If there is a record of this depth, the author may well include the figures in his closure, not only for the purpose of enabling a better understanding of all the conditions at the site, but also as a contribution toward the general knowledge of the depth to which these rivers cut their valleys before the coming of the glaciers.

On pages 1270 and 1271, the author has referred to the movement of the dam both vertically and horizontally in a down-stream direction as the load of the water came on it. The data presented are interesting and instructive. Buttress No. 6 apparently settled vertically 0.40 in., notwithstanding the fact that it was founded on rock. Now this 0.40 in. may represent either shrinkage or compression of the concrete, or both. The author has not referred to this possibility and neither has he made mention of the cracking of the concrete which the settlements and lateral movements as recorded would seem to indicate. Referring to the motions which have taken place, he states that they show little change in either settlement or alignment, and that the dam appears to be at rest with the exception of changes due to temperature. If there are any data available on the effect of changes due to temperature, the author will perform a most valuable service by presenting them, because the ultimate life and integrity of nearly all engineering structures are, in great measure, dependent on the daily and seasonal motions which result from ever-recurring temperature changes.

The profile in Fig. 18 shows very clearly the variation in pressures from the line of the cut-off down to the tail-water, while Fig. 19 shows the fluctuation in uplift pressure as observed in different seasons. The author's second conclusion from the data presented is to the effect that an increase in the temperature of the water, having the effect of decreasing the resistance to flow through the sand, not only increases the velocity, but also decreases the loss of head. With this the speaker does not agree and presents the following hypothesis.

As the temperature of the water rises the volume of each sand grain increases and the volume of the voids is correspondingly decreased. In the smaller passages thus formed the static pressures would necessarily rise, it being assumed that the head of water in the pond remains constant. Conversely, during the winter season, the voids in the sand are at their maximum. The water then flows with more facility and the uplift pressures decrease. A lowered hydraulic gradient, as shown by the pressure pipes, indicates that either less water is finding its way into the sands or that the loss of head within the sand has been reduced. There is no information available as to changes in the quality of leakage from summer to winter, but the speaker is willing to hazard the estimate that the leakage through the sand is greater in winter than in summer notwithstanding the greater viscosity of the water due to the lower temperature. The so-called uplift pressure is not a direct index of the quantity of water which passes under the dam. The leakage must be in inverse proportion to the uplift pressure.

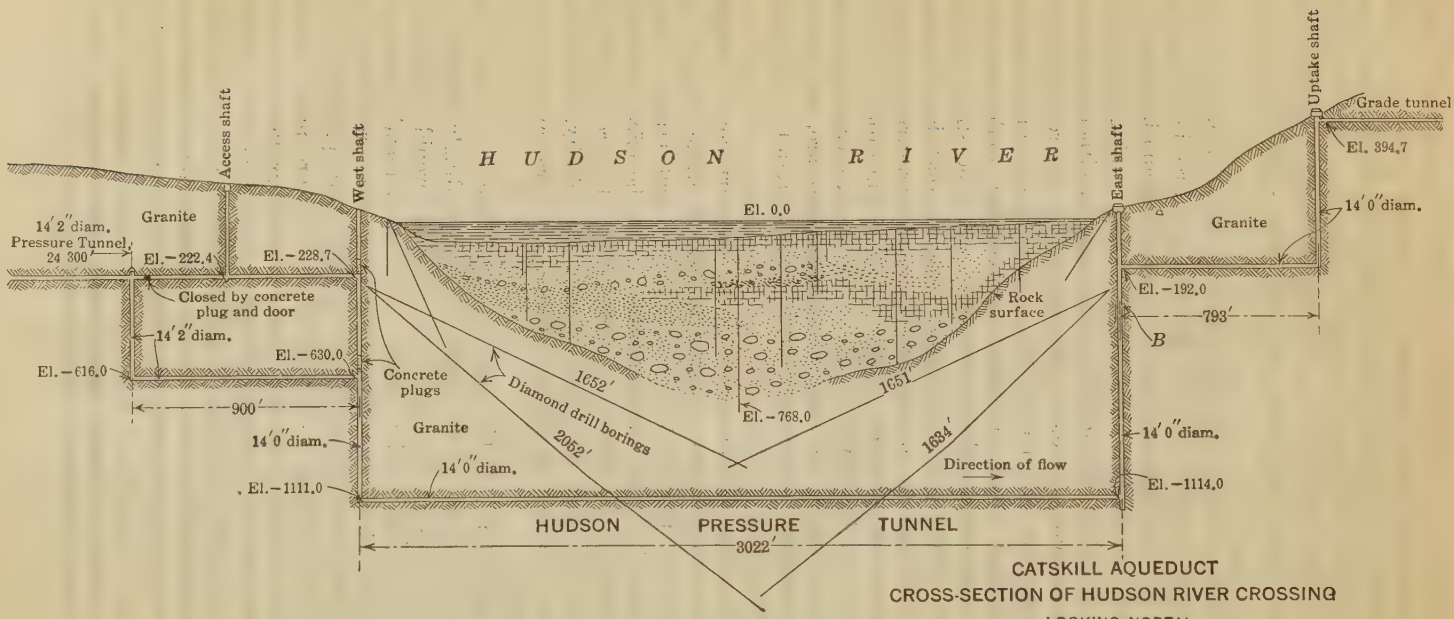
Access shaft
West shaft
HUDSON RIVER
East shaft
Uptake shaft
Grade tunnel
El. 394.7
14′ 0″ diam.
Granite
14′ 2″ diam.
Pressure Tunnel
24 300′
El. −222.4
El. −228.7
Closed by concrete plug and door
14′ 2″ diam.
El. −630.0
El. −616.0
900′
Concrete plugs
Diamond drill borings
1652′
2052′
1651′
1634′
El. 0.0
El. −768.0
Rock surface
El. −192.0
793′
B
Granite
14′ 0″ diam.
14′ 0″ diam.
El. −1111.0
Direction of flow
El. −1114.0
HUDSON PRESSURE TUNNEL
3022′
CATSKILL AQUEDUCT
CROSS-SECTION OF HUDSON RIVER CROSSING
LOOKING NORTH

FIG. 22.

Nearly all experiments on the flow of water through sands have been made under conditions in which the sands themselves were not confined within a definite volume. In the case under discussion there is a body of sand on which is placed a large dam with much weight of water above it. In a filter bed the upper surface of the sand is always free to move as the sand grains change their volume with changes in temperature.

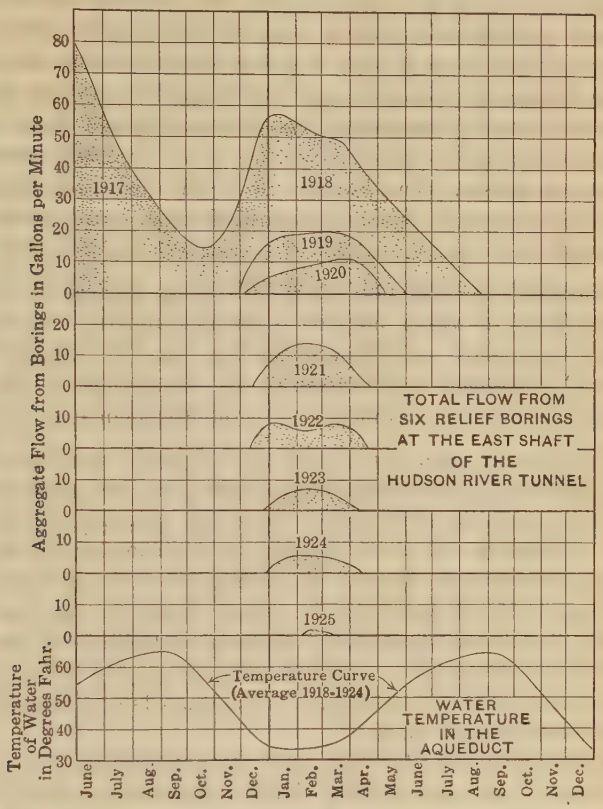
Aggregate Flow from Borings in Gallons per Minute
1917
1918
1919
1920
1921
1922
1923
1924
1925
TOTAL FLOW FROM SIX RELIEF BORINGS AT THE EAST SHAFT OF THE HUDSON RIVER TUNNEL
Temperature of Water in Degrees Fahr.
Temperature Curve (Average 1918-1924)
WATER TEMPERATURE IN THE AQUEDUCT
June July Aug. Sep. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sep. Oct. Nov. Dec.

FIG. 23.

On Fig. 22 is shown a cross-section of the Hudson River at the point where it is crossed by the Catskill Aqueduct Tunnel. Shortly after the tunnel was filled with water to the hydraulic gradient, which is 425 ft. above the river surface, a series of cracks developed in the concrete lining of the shaft near the point marked *B*. These cracks were the result of rock movement caused by the escape of small quantities of water into the joint planes of the rock. In this manner pressure was applied over large areas and the rock moved. The action was analogous to the well-known use of compressed air for the quarrying of granite.

In order to control this action and prevent the possibility of further movement, six diamond drill holes were put down around the shaft from the

surface to a depth of 500 ft., the purpose being to intercept the rock seams and so to provide them with drainage and thus prevent the accumulation of pressures within the mass of the rock. Observations have been made on the quantity of flow from these holes from the time they were bored to the present. The results are shown in Fig. 23.

It is of interest to observe (*a*) that the reduction in the total leakage has been progressive from year to year; (*b*) that the tendency has been for the leakage to begin a little later and to cease a little earlier each year; and especially (*c*) that the leakage is always greater in winter than in summer.

The greater leakage in winter in this case is due to the same cause which acts to reduce the uplift pressures at the Sherman Island Dam during the colder months, namely, an increase in the size of the channels through which the escaping water passes. The increase in uplift pressures which accompanies a rise of temperature cannot be ascribed to a cause which "* * * not only increases the velocity, but also decreases the loss of head."

HENRY GOLDMARK,* M. AM. SOC. C. E.—In heavy structures, such as the Sherman Island Dam, of which one part is on rock and the adjacent part on a more or less compressible soil, the speaker has found it difficult if not impossible to avoid settlement and cracking. He would like to inquire what means were taken to avoid these difficulties in the Sherman Island Dam.

BYRON E. WHITE,† M. AM. SOC. C. E.—Referring to the curves (Fig. 19) showing the hydrostatic heads or pressures in the three test pipes beneath the dam down stream from the sheet-pile cut-off (Fig. 18), the curves shown by the author in presenting the paper indicate a condition of affairs which is not revealed as satisfactorily by Fig. 19, namely, that the pressure heads in the pipes during 1924 are markedly less than those for the corresponding period in 1923. To be more specific, the maximum pressure head was reduced from about 13 ft. in 1923 to about 11 ft. in 1924, or roughly 15 per cent. This indicates that considerable silting behind the Sherman Island Dam and perhaps beneath it has already taken place.

The curves presented by Mr. Merriman showing progressive decrease in the seepage at one of the shafts of the Catskill Aqueduct probably are representative of the condition to be expected at the Sherman Island Dam. There is, however, one notable difference, namely, that in the case cited by Mr. Merriman the maximum seepage occurred during the cold months, while at Sherman Island the minimum occurs during those months.

Unless some unusual condition arises, it seems fair to conclude that a progressive decrease in pressure will take place as time goes on and, if so, this will confirm the thought that after a period of years, very little, if any, hydrostatic pressure beneath the dam will be shown by the readings taken at these pipes. If such proves to be the case, a knowledge of the facts will do much to justify designing engineers in adopting what now seems a bold design for use in similar difficult situations.

* Cons. Engr., New York, N. Y.
† Engr., Utica Gas & Elec. Co., Utica, N. Y.

It is hoped that in his closing discussion, the author will bring the curves down to date, including the latest reliable observations, as this will complete two annual cycles of variations, which, if in accord with the speaker's assumption, will further confirm his conclusion. It is also hoped that these observations will be kept up and, at a later time, that they will be made available to the profession as a guide to the design of future structures of this character.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—The multiple-arch section of the Sherman Island Dam founded on sand is decidedly interesting and the problems encountered seem to have been carefully studied. To have constructed a gravity dam of equal safety would have increased the cost possibly 100%, or even more.

Referring to Table 2 the writer does not understand whether the arches were built at the same time as the buttresses, or what provisions, if any, were made to permit a re-adjustment of the various parts of the dam without producing internal stresses. Evidently, since the measurements of movements of the multiple-arch dam were made, the author expected such movement.

Referring to Fig. 11, and scaling the position of the resultant, it appears that uplift was not taken into account. Also, it would have been instructive to have had some actual tests as to whether or not uplift existed under the gravity section.

It seems to the writer that the solution by means of a multiple-arch dam is very much to be preferred to either a gravity type or a dirt dam, but he believes that perhaps some provision for permitting slight movements of the various parts would have been desirable.

CHARLES W. COMSTOCK,† M. AM. SOC. C. E. (by letter).—The writer has found Fig. 19 which shows the observed relation between water temperature and sub-pressure, to be of particular interest. Those who mentioned this point in the oral discussion, with one exception, explained the phenomenon by reference to the known variation of viscosity of water with temperature, and dismissed it without further comment. At first thought, the same idea occurred to the writer, but further consideration forced the rejection of this explanation as inadequate.

The absolute viscosity of pure water at 0° cent. (32° Fahr.) is 0.017928 dynes per sq. cm., and at 20° cent. (68° Fahr.), it is 0.01006 dynes per sq. cm. For a constant head the velocity of flow through a given capillary tube or network of capillaries is inversely as the absolute viscosity. It would seem, therefore, that the volume of water flowing under the dam in winter should be 56% of that in summer; but it is not clear how the piezometric head can be affected except to the extent of the difference in velocity heads. The velocity of flow in the sand probably does not exceed 8 or 10 ft. per day, so that the effect on the sub-pressure of any change in velocity would not be measurable. To the extent that the sub-pressure is affected by this cause, however, it must be

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† Engr., Dwight P. Robinson & Co., Inc., New York, N. Y.

the reverse of that shown in Fig. 19, the pressure being higher in winter when the velocity is less.

It seems much more probable that the observed variation in sub-pressure is due to the change of elevation of the tail-water. Perhaps, if sub-pressure and tail-water elevation were plotted together, they would show the same synchronism as is shown in Fig. 19. The author states:

"The elevation of the water surface in the pipes varies closely with changes in elevation of the river surface just below the dam, while changes in pond elevation have less relative effect."

The apparent relation between sub-pressure and temperature may be only incidental and due to the fact that the winter season is the period of low tail-water.

The author states that the design considered sub-pressure as acting on the entire base, although the pressure probably acts on less than 50% of the area because of sand contact.

The writer believes that provision for sub-pressure on the entire base, as in this design, is correct—not ultra-conservative. The view that sand contact can protect against sub-pressure is illusory. Pressure on the under side of a sand grain in contact with the structure has the same effect as water pressure directly against the concrete base. If all the sand grains were equal prisms and were arranged so as to form a series of prismatic columns extending from the lower side of the structure to an impermeable rock foundation at any distance below, there might be some excuse for reducing the area subject to upward pressure; but with the haphazard arrangement of sand grains of all shapes and sizes, approximating much more nearly to a group of packed spheres than to prisms with vertical axes, any imagined protection against upward pressure by sand contact is too uncertain for use as an element in design. Volume porosity is not a measure of the area subject to pressure. For example, consider a mass of equal spheres with the closest possible packing. The porosity is only 26%, but the entire area of the base of a structure resting on such a mass would be subject to full upward pressure of any fluid filling the pores. It is true that a structure weighing 3 tons per sq. ft. cannot rest on geometrical points; but it can be supported by an actual area of contact of less than 1 sq. in. for each square foot of base.

The experiment described by Mr. Meem, in which he found a greater pressure necessary to raise a piston surrounded by sand than one immersed in water only, is susceptible of an explanation other than that of reduced area of contact. Had Mr. Meem made a number of observations with the piston at various depths in the sand, he might have found the increased pressure accounted for by friction of the sand on the sides of the piston. Further experiments with Mr. Meem's apparatus would be of great interest.

Water-proofing of the face of the dam is of vital importance. The best of concrete is not water-tight and, lacking a water-proof face, will become saturated. The arches are thin (only 2 ft. thick) and are subject on the downstream side to temperatures below freezing throughout the winter. Thus, the absorbed water will freeze, and the concrete will be subject to a slow but certain and powerful disruptive effect. The writer knows of one important multiple-

arch dam in which this action proceeded to the danger point in the course of a few years.

The author says: "Before the dam was closed, the northern half was given four coats of 'Ferritex' water-proofing." If this material proves effective he has provided against the destructive action mentioned. No statement is made concerning the southern half of the structure. It would be interesting to know whether some other water-proofing process was applied to that part or whether for some reason it was left untreated.

The author makes no reference to calculated stresses in the piers. It would be a matter of interest if he would outline his analysis and give the resulting stresses. The writer has recently examined an important multiple-arch dam in which every pier showed an open crack beginning near the face about the middle of its height and extending approximately perpendicular to the face nearly to the down-stream ends of the piers. These cracks gave every evidence of resulting from excessive tension or shear—perhaps both. In that structure the face of the dam made an angle of about 50° with the horizontal except near the top. The cracks were not due to settlement, for every pier was founded on solid rock. They were probably not temperature cracks, for the climate is equable and freezing is unknown. The conclusion that they are stress cracks seems unescapable.

The report of the Swiss engineer, Stucky, on the Gleno Dam failure in Italy revealed by calculation dangerous shearing stresses in the piers on planes making an angle of about 50° with the vertical, and observations on some of the broken piers showed actual failure along these planes. The danger lines calculated and observed by Stucky corresponded closely to the cracks already noted by the writer.

The peculiar profile of the piers in the Sherman Island Dam may result in stresses radically different from those in the more usual type, and for this reason the author's analysis would be all the more welcome.

Walter H. Wheeler,* M. Am. Soc. C. E. (by letter).—This paper presents a description of some interesting hydraulic and structural engineering work. The dam is especially interesting because of the foundation conditions and the design and execution of the structures to meet these conditions. Judging by all the evidence submitted it would seem that it may be regarded as safe and substantial as long as no scouring occurs under the dam. It also appears from the data given that the possibility of any such scouring is being reduced each year.

The writer is much interested in the design of reinforced concrete dams, and believes that when well designed and constructed they may be regarded as permanent structures, provided maintenance is not too flagrantly neglected. There is sufficient experience with hydraulic structures of reinforced concrete to establish this as a fact. It is the writer's experience, however, that concrete subjected to the action of water and frost offers much better resistance to such action when its fine and coarse aggregates, in addition to all the recognized requirements, are materials which themselves will not be disintegrated by frost, and have been washed clean of all clay and loam.

* Designing and Cons. Engr., Minneapolis, Minn.

In Fig. 24 (*a*) is shown another type of reinforced concrete design* which might have been used for the retaining wall section of the Sherman Island Dam, and which, the writer believes, would have reduced the concrete yardage required in that part of the dam more than 25% without sacrificing the strength, stability, or efficiency of the structure.

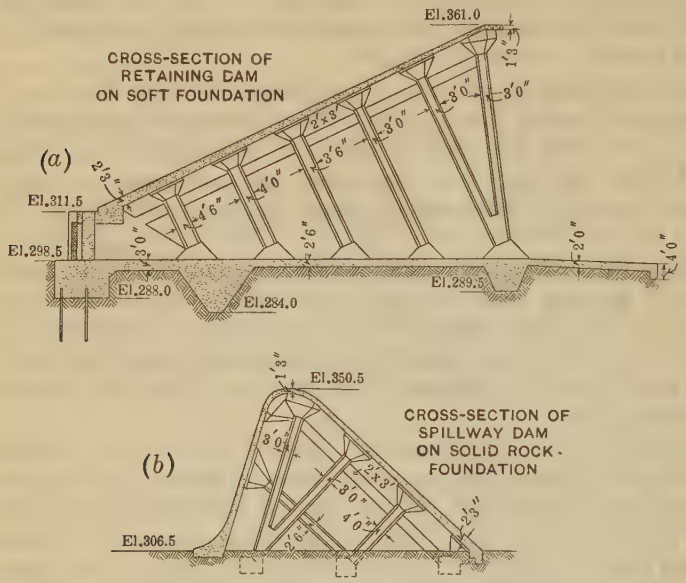

FIG. 24.

Likewise, in Fig. 24(*b*) is given a reinforced concrete design* of a similar type for the spillway part of the dam. The deck slab is carried over the downstream face to form the spillway and the floor slab is omitted, as it is not required where rock can be reached for the foundation, since the columns can be supported on footings which are designed to take the thrust and are built into the rock. When the rock is stratified and flat, it is advisable to tie these footings together with a strut at the rock line extending from the heel of the dam to the outermost footing.

The writer believes that this reinforced concrete design for the spillway part of the Sherman Island Dam would require only about 30% of the material for the gravity dam as built. Considering the cost per cubic yard of concrete for the reinforced structure complete to be approximately 2¼ times that for the gravity type, the net saving in cost for the reinforced design, as compared with the gravity type structure, would therefore be between 30 and 35 per cent. If designs similar to those of Fig. 24 had been used for both the retaining wall and spillway sections, the cost of the whole dam might have been reduced as much as 25 per cent. The possibility of so great a reduction in the first cost of a structure would warrant consideration of this method in similar cases. It is based on the principle of a flat slab with column supports.

* Patent No. 1,351,588 filed November 23, 1918, Serial No. 263,855.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).—The members are greatly indebted to the author for his complete description of this project, which gives a valuable example to be followed in designing engineering structures. In addition to its wealth of information in such compact form, the paper makes interesting reading not only to those actively engaged in hydroelectric work, but also to the average engineer. The writer, in studying this paper, was especially interested in the parts dealing with the design of the substructures and with the control of soils for foundation material.

The observed variations in the upward pressure on the base of the dam (pages 1287 and 1288) are interesting. The writer, however, cannot agree with the author's explanation that the fluctuation is caused by the variation of the river temperature and that the two variables are connected by a simple relation. As shown in Fig. 19, the maximum temperatures of the river occurred about August 1, 1923, and August 1, 1924, whereas the maximum upward pressures occurred about July 1, 1923, and July 15, 1924. The minimum temperature was about February 15, 1924, and the lowest pressure, January 15, 1924. In other words, admitting the author's explanation, the pressure change under the bottom of the dam occurred one month before the change in temperature of the river water. This should make a good method for predicting the temperature of the river a month in advance. Actual temperature measurements will show that the ground-water (and the soil) beneath the dam does vary in temperature to a small extent, but that the variation lags behind that of the river temperature by at least a month.

It should be remembered that the water seeping through the soil from the river above the dam to the tail-water, in accordance with the laws of thermodynamics, tends to gain or lose heat (depending on whether the water is cooler or warmer than the soil) until its temperature is the same as that of the soil. It would be interesting to see a more complete analysis of the data collected on this phenomenon.

Until quite recently, in the absence of data on the subject, it has been assumed that there is direct proportionality between the test area and the bearing value of the soil tested, in other words, that the unit value is independent of the size and shape of footing. For the determination of the soil values at this site, the author (page 1282) described the use of a test area (1.737 sq. ft.) which showed a soil-bearing power 2.9 times the unit weight of the completed structure on its footings. The first real experimental work on this important subject has recently been reported† by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., who shows that, as long as the soil action is largely elastic and the deformations remain small, if like intensities of load are applied to bearing blocks of different sizes, the relative penetrations of these areas into the soil vary as the square root of the areas; that is, a large area will penetrate more than a smaller area under the same load intensity. Although the tests do not include very large areas, they are extensive enough to show that the present assumptions are false. In the light of this new research work, the author should modify his conclusions from the experiments shown in Fig. 16.

* Structural Engr., New York, N. Y.

† *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 893.

The writer has never been able to discover, in spite of considerable questioning on the subject, why certain recently developed methods are used extensively in foreign countries for the expressed reason that they tend toward economy in construction, whereas the same methods are practically unknown in this country. The difference in the relative cost of materials and labor in the United States and abroad is not a sufficient reason.

The author has described (page 1283) the various expedients used in building the substructure for the power-house. Trouble was chiefly due to the presence of large quantities of water in the soil, creating boils of dangerous size. An ingenious, yet simple, method of overcoming such difficulties seems to be in quite common use in Europe, especially in Germany, where sub-surface structures are almost always in water-bearing silt or similar soils. In "Die Grundwasserabsenkung in Theorie and Praxis"* Dr.-Ing. Joachim Schultze describes numerous actual constructions of substructures, and gives a detailed theoretical analysis, using a method by which the sub-surface water was temporarily removed from the region of activity by enclosing the area with rows of vertical pipes from which the water was steadily pumped. The spacing of the pipes and the power of the pumps controlled the amount by which the ground-water level was lowered.

The practical examples cover (1) the building of subways and foundations by coffer-dam methods in the middle of streams; (2) in similar locations where a water-tight roof is first built and tunneling methods are used without compressed air; and (3) the excavation of foundations in water-bearing soils with and without the use of sheet-piling. In all these cases, the excavation and masonry work was in the dry, the ground-water being lowered in one case as much as 10.25 m. The method is certainly general enough to attract more attention than it has in the United States.

D. C. Henny,† M. Am. Soc. C. E. (by letter).—The Sherman Island Dam at maximum pond elevation resists a water head of about 53 ft. Its most interesting feature is its sand foundation, and the author's information regarding underflow and uplift is decidedly valuable.

The importance of uplift is far less for a dam with a sloping water face than for one with an approximately vertical face. The importance of a flat subflow water gradient as expressive of the velocity of underflow and the danger of piping is equally great in both cases.

With a homogeneous foundation and a given head the grade can be reduced only by enforcing long water travel. In the case under discussion three means were used to accomplish this end, namely:

(*a*) A double row of sheet-piling, together with two cut-off walls;
(*b*) A blanket of sand-fill on the down-stream apron; and
(*c*) An up-stream sand blanket.

The sheet-piling appears in this case to be the most effective. If the reduction of water pressure under the foundation were to be estimated on the "line-

* Reviewed by the writer in *Engineering News-Record*, October 16, 1924.
† Cons. Engr., Portland, Ore.

of-creep" theory, the total length of travel would be about 404 ft., along which head of 53 ft. is destroyed, or a ratio of 1 to 7.62.

The pressure loss as far as the up-stream test pipe which compels 228 ft. of creep, would be estimated on the basis of a uniform gradient at 30 ft., while the measurement in the test pipe shows it to have been 41 ft. This high loss is the more striking as the distance of creep was estimated by following both rows of sheet-piling up and down, although the distance between the two rows was only 6 ft. and the friction due to water movement through the sand across this distance might easily be far less than that following the line of creep.

The relatively high beneficial effect of the sheet-piling indicates that the reduction it causes in the gradient of the underflow is not the simple result of increasing the water travel, but probably of enforcing it partly in a vertical direction.

The author's description does not indicate whether the coarse to fine foundation sand showed variation of fineness in an erratic manner or whether approximately horizontal strata existed of various degrees of fineness. The latter is generally the case in water-deposited sand, and frequently thin layers of silt or clay are found interspersed between the sand strata. The benefit of breaking up the horizontal water movement in such a case by sheet-piling is obvious. Sheet-piling likewise renders piping directly under the foundation across its entire width impossible.

The use of sheet-piling as an economic measure for reducing the water movement and the danger from piping is, therefore, usually justified. Other means, however, are at hand which may accomplish the same general result, and which at times will be less costly and equally satisfactory and safe, especially where the sand foundation is homogeneous.

Two of these means were used as additional measures of safety in the Sherman Island Dam. The down-stream blanket lengthens the water travel by building up a counterhead against the tendency of any springs to break out directly down stream from the dam. In this case, the sand-fill also provided a load on the concrete apron, preventing it from being raised by uplift.

Any down-stream blanket, however, has the effect of increasing the uplift under the main dam, and, from that point of view, it is objectionable, especially in the case of a gravity dam. In the present instance this objection is relatively small. The only question that arises concerns the object of the concrete part of the apron, the omission of which would apparently have had no effect on the conditions controlling the underflow and the safety of the dam.

An up-stream blanket has the double advantage of lengthening the water travel and of reducing the uplift under the dam. To be fully effective, it should be practically water-tight, and its connection with the main dam should leave no unsealed crack or joint.

A sand blanket can hardly be expected to be satisfactory unless carried high up against the dam. Water from the pond gains access to the foundation under the toe of the dam from many directions and distances. Without any blanket, its pressure is equal to that resulting from the full pond elevation. A 20-ft. sand-fill, such as was used at Sherman Island, cannot be expected to cause a reduction of pressure at the up-stream toe greater than would result

from 20 ft. additional travel through the sand of the foundation, even though the blanket has a length up stream of 75 ft. and more. This is because the pressure reduction which might have occurred with a 75-ft. tight blanket is partly neutralized by vertical seepage through 20 ft. of sand. It would seem, therefore, to the writer that a lesser length of sand blanket would have been equally effective.

In his discussion of the paper by James B. Hays, Assoc. M. Am. Soc. C. E., on "Designing an Earth Dam",* the writer referred to the use of a thin continuous slab of concrete as constituting a type of up-stream blanket which, if well connected by steel to the main dam, is likely to give the greatest benefit in relation to its cost where a masonry dam is founded on a homogeneous permeable foundation of great depth. The thickness need not exceed from 3 to 6 in. if proper steel reinforcement is used and a fairly even and well-settled bed can be readily secured. If "gunite" is used, even a lesser thickness would suffice.

Regarding the actual effect of uplift on the loading of the body of the dam, statements are frequently made that the uplift probably acts on only a part of the foundation due to contact between the dam and the soil. The author estimates this contact at 50 per cent. It is not believed, however, that any material reduction from such a percentage can be safely assumed. If sand is examined miscroscopically, it is seen that the grains have an irregular shape and that two particles in contact touch only at a few points with extremely small contact surfaces.

Settlement by water will tend to fill large voids with smaller sand particles, but actual points of direct contact will remain small. Sand derives its satisfactory nature as a foundation from the fact that generally it has settled in water, that its individual grains are very hard, and that little settlement occurs on loading, as particles under ordinary loads do not crush and, except in the case of quicksand, do not move sidewise.

Thus, the areas of actual contact between loaded particles of sand are relatively small. It is possible, of course, that more perfect contact occurs between concrete and sand than between sand and sand, but this would not affect the uplift, as the particles of sand gripped by the foundation concrete must rest on other sand particles and the uplift reduction by contact would still be measured by the relative area of contact between sand and sand.

If such estimates as 25 and 50% reduction in uplift due to contact are to be used in designs, this subject acquires sufficient importance to justify experimental research, which should also cover contact with clay and with solid rock.

With clay the conditions are dissimilar from those with sand, but not because of the lesser angularity of grains, which does not affect the extent of the possible uplift materially, as even with perfectly spherical shapes the aggregate area of contact must be small. Neither does the smaller size of grains necessarily alter the uplift although it may greatly diminish the rate of water movement. The essential difference lies in the presence of colloids that may produce practical or even absolute water-tightness, but that, on the other hand, by progressive yielding under the effect of a unit water pressure greater than the unit compression to which they are subject, permit dangerous creeping

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 58.

along the planes of contact. It is this fact that makes clay an unreliable material for back-filling coffer-dams and makes it dangerous to depend on the reduction of uplift by reason of contact.

In the case of a tight rock foundation, however, conditions may well be essentially different, and large percentage reductions on account of the contact may be fully warranted. No means are at hand, however, on which to base a safe estimate of this reduction.

The relation indicated by the author between the temperature of water and the uplift is interesting and valuable. He deserves high commendation for the comprehensive scope of his paper and the clearness of its presentation.

E. W. Lane,* Assoc. M. Am. Soc. C. E. (by letter).—The author's measurements of the upward pressure on the multiple-arch dam are of particular interest. It is the most valuable record of such experiments which the writer has seen, but he regrets that Mr. Parsons did not give more complete data, or make the experiments the subject of a separate paper.

The writer, with E. L. Chandler, M. Am. Soc. C. E., made some experiments on upward pressure against a small dam on a gravel foundation at Dayton, Ohio.† One of the surprising results was the discovery that gas collected under the dam. It was inflammable, probably methane, formed by the decomposition of small quantities of vegetable matter in the gravel of the river bed. The collection of gas beneath a dam is detrimental, as it exerts a buoyant force, similar to an upward pressure, the amount being equivalent to the head due to the thickness of the gas layer. The dam at Dayton had concrete cut-offs similar to those at Sherman Island, and although the base was poured in sections, it seemed to be gas-tight. At the Sherman Island Dam, gas could collect between the concrete cut-offs, being confined at the ends by the rock ledges, without showing in the pressure pipes, the ends of which were at a lower elevation. It appears possible, therefore, that in some places the effective upward pressure may have been as much as 5 ft. more than that indicated by Fig. 18. It would appear to be much easier to vent a hollow dam than one of solid masonry.

Fig. 19 shows surprisingly little decrease in pressure during the second year over that measured in the first. The pipes in the dam at Dayton were installed in the summer of 1917, but, due to the World War, no observations were made until the summer of 1919. In this interval, a thick layer of mud had formed above the dam, and little upward pressure existed, even above the up-stream row of sheet-piling. The pressure that did exist seemed to be due to ground-water, as the temperature of the water was considerably below that of the river. At another dam on the same stream, pipes were installed, but no measurements were made until a year or more had elapsed. There was almost no upward pressure, although gravel was being excavated from the river bed a short distance up stream, and no doubt cut through the mud layer. This river drained an alluvial country, but it did not carry an unusual quantity of silt.

* Detroit, Mich.

† *Engineering News-Record*, Vol. 80 (1920), p. 1014.

HOWARD M. TURNER,* M. AM. SOC. C. E. (by letter).—This paper describes a power development in which great natural difficulties were overcome. Among the most interesting features is the design of the spillway, used to avoid the necessity of a high overflow dam on an earth foundation. This discussion will be confined to the hydraulics of this spillway and its wasteway channel.

For many years it has been considered that lengthening a spillway by inclining it up stream, across a river where the water is moving straight down stream, is of doubtful benefit. An engineer, noting the Sherman Island Spillway design that not only depends for its length on long stretches up and down the stream but is located on one side of the main thread of the river, immediately wonders whether the old principle was entirely wrong.

With this in mind, the writer has made a general analysis of some conditions that will occur on the Sherman Island Spillway. The figures must be taken as more or less general, as exact data such as elevations, dimensions, etc., were not available; they are, however, sufficiently close to bring out the points in question. The conditions outlined should be expected to take place only under circumstances of very large flow and probably would not obtain during any small or even moderate freshet where the velocities involved are much lower.

The spillway is 864 ft. long and its capacity is given as 100 000 cu. ft. per sec., with a pond surface of 8.75 ft. above the crest. To obtain this discharge, allowing for velocity of approach to the weir as a whole (Section *C-D*, Fig. 25) of about 2.3 ft. per sec., requires a coefficient in the Francis weir formula of 4.03. This seems to be a high coefficient. The coefficient for this shape of spillway found by the U. S. Geological Survey (Cornell Experiments) was 3.88 with the curve of coefficient practically level at 4 ft. over the weir.†

The question of whether the whole length of this crest is 100% effective is more serious. To obtain this result each part must discharge its full share, and if the discharge over any of it is reduced in any way the capacity of the spillway will be less. In attempting to analyze this, the length of the spillway was divided into twenty sections (Fig. 25). While the conditions of approach to Sections 1 to 13 (the latter about the center of the curve) are good, the remainder of the dam (Sections 14 to 20) comprising more than one-third of the length, must receive its water through a shallow curved channel at high velocity.

If the whole spillway is effective, each foot of crest taking its share (115.7 cu. ft. per sec., with a total flow of 100 000 cu. ft. per sec.), it would require conditions of flow somewhat as shown in Fig. 25. It is assumed that all the water will flow past Section *A-B* in the main river above the spillway with a uniform velocity of about 2 ft. per sec., and will be distributed so that each section of the dam will take its share. On this basis, 35 000 cu. ft. per sec. would have to go past Section *E-F* where the velocity required will be 8 ft. per sec.‡ Beyond Section *E-F*, in order to get the best results, the whole

* Cons. Engr., Boston, Mass.

† "Weir Experiments, Coefficients and Formulas," by Robert E. Horton, M. Am. Soc. C. E., *Water Supply Paper No. 200*, U. S. Geological Survey, Plate XXV.

‡ Average elevation of bottom assumed = 341.

width of the channel is assumed to be effective. Even with this distribution the average velocity (assuming a bottom elevation of 344 and allowing for the slope due to friction) would be about 8.5 ft. per sec. and about 9 ft. per sec. just up stream from the dam. The average friction loss (with $C = 60$ in the Chezy formula) would be 0.7 ft. and the elevation of the water at the crest at Section 17-18 would be less than 7 ft. instead of 8.75 ft. Clearly this part of the spillway cannot take its full share without a higher level of pond surface.

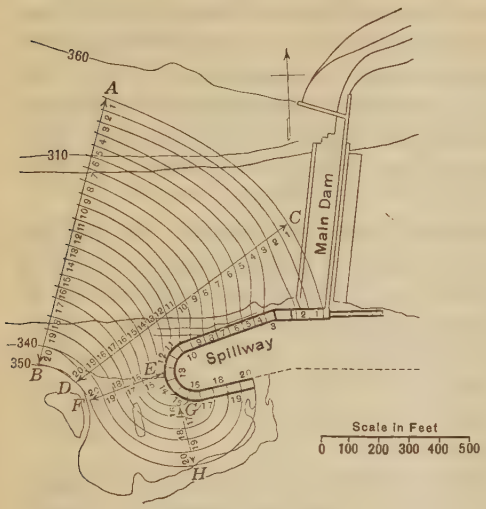

FIG. 25.

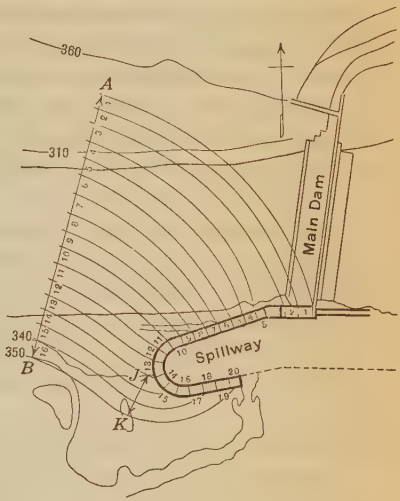

FIG. 26.

It is, however, almost impossible to conceive of a flow toward this part of the weir as assumed in Fig. 25. Fig. 26 shows in a general way more nearly what is to be expected at this point; that is, that the water will flow toward the spillway in a down-river direction inclined to the crest line. As it is unlikely that water, flowing with the velocity required for any large flow over this end of the spillway, can make a sharp turn in approaching the dam, it will cross the crest in very much the same inclined direction, cutting down the effective length of the spillway.

The writer has observed a case over a low part of the dam on the Connecticut River, at Turners Falls, Mass., similar to this, except that the spillway is much less inclined to the general direction of flow in the river. During very high water it is clearly noticeable that the flow over this spillway is in a "down-river" direction inclined to the crest line.

With the water flowing as shown in Fig. 26 the effective length of this part of the spillway, including Sections 14 to 20, may be assumed to be about 160 ft., the high velocity preventing any appreciable change in the direction of flow. With the same assumptions of bottom elevation and of the coefficient, C, this will mean a head on the crest at Sections 17 and 18 of about 7.8 ft., the total flow over these seven sections being about 15 000 cu. ft. per sec., requiring a velocity at Section J-K of 7.2 ft. per sec. Thus, this part of the

dam will discharge only 43% of its share, and the discharge of the whole will be cut down by about 20 per cent. On this basis it will require a pond level of about 10.9 ft. above the crest to discharge 100 000 cu. ft. per sec.

As the pond above the dam fills up with silt, which in time is probable, so that the velocity of approach with flood water is high, somewhat the same conditions of flow are likely to occur on the other straight leg of the spillway, Sections 1 to 10, further cutting down its capacity.

Referring now to conditions in the wasteway channel below the spillway, it is stated that the water will flow 18 ft. deep, with a velocity of 31 ft. per sec., at a point opposite the main dam. This prediction requires assumptions which, if made incorrectly, may entirely change the conditions. Taking into account the high velocity, the changes in velocity, and the shocks and interference to orderly flow through this channel caused by the discharge from the spillway along its sides, these assumptions become very difficult to make with any degree of certainty.

For example, the total head between the water surface above the spillway and the water surface in the wasteway channel opposite the main dam is about 26.5 ft. The velocity of 31 ft. per sec., given at the lower section requires 15 ft., leaving 11.5 ft. to overcome the losses due to friction, sudden changes of velocity and direction, the losses caused by the interference of the flow from the spillway on the sides, etc. Friction may take 5 ft. of this 11.5 ft. (with C in the Chezy formula = 50) leaving 6.5 ft. for other losses, or only about 0.4 of the velocity head.

Suppose that, due to the plan of the spillway and the greater quantity of water coming into one side of the channel than the other, this channel is only 80% effective, that is, that practically all the water may flow through 80% of the width—a perfectly possible case, often observed under much simpler conditions of flow and velocity. In this case the velocity instead of being 31 ft. would be 38.8 ft. per sec., and the head required for this velocity would be 23.4 ft., leaving only 3.1 ft. for all losses. A condition such as this would submerge the spillway at its upper end.

Then, again, the conditions of flow may be modified by the coefficient chosen for the friction loss. A value for C of 40 in the Chezy formula will cause a slope of the water surface steeper than the bottom slope; a value of 60, one much flatter. The selection of a value for C with a velocity of 31 ft. per sec. for a channel of this size cannot be made with the same confidence as for the lower velocities that are backed up by a great body of experimental data.

Observations of the behavior of this spillway and its wasteway channel during a great freshet should add greatly to engineering knowledge as to the practical hydraulics of the wasting of flood flows.

Fred A. Noetzli,* Assoc. M. Am. Soc. C. E. (by letter).—The Sherman Island Dam as described by the author is an interesting example of a multiple-arch dam built on a pervious foundation.

* Cons. Hydr. Engr., Los Angeles, Calif.

In general, it is considered essential that high dams of the multiple-arch type be founded on rock, inasmuch as irregular settling of the buttresses would introduce deformations and stresses in the arches which might be greater than those produced by the water load. With a heavy foundation slab, a close spacing of the buttresses, and other proper precautions, it would seem to be perfectly safe to build multiple-arch dams of moderate height on foundations which may be subject to slight settlements. On a rock foundation the settlements are too small to affect appreciably the stresses in such a structure.

The measurements made by the author for ascertaining the uplift under the foundation slab, and especially for determining the horizontal and vertical settlement of the buttresses, are of particular interest. It is hoped that the measurements are being continued and that the author will give in his closure additional data on these conditions and also on the effect which the relative settlements of the buttresses had on the arches.

The Sherman Island Dam is remarkable also for the flat slope of the arch barrels which in the lower part of the dam are inclined only 22° 40′ with the horizontal. A solid gravity dam would have involved large quantities of concrete to provide the weight necessary for a safe factor against sliding. With the multiple-arch type of dam as designed by the author, the weight of the water overlying the arches helps to prevent the dam from sliding. The economic advantage of the multiple arch over the gravity type is, therefore, self-evident.

The buttresses of the Sherman Island Dam were apparently not reinforced. Although the thickness of the buttress walls appears to be ample to carry the loads imposed on them by the arches, it is possible that shrinkage and temperature deformations will produce cracks which could have been reduced to the size of hair line cracks by adequate steel reinforcement. Such cracks have been observed on a number of multiple-arch dams in California. Although theoretical investigations regarding shear and inclined tension stresses have shown that the cracks in these dams are not due to the water pressure, modern practice nevertheless favors reinforcing the buttresses in a horizontal and inclined direction for temperature and shrinkage deformations. In the multiple-arch dam at Palmdale near Los Angeles, Calif., the writer observed inclined hair line cracks in the buttresses before any water had accumulated in the reservoir. The buttresses of this dam are strongly reinforced, but some small shrinkage cracks showed during the winter months even before the completion of the dam.

In another high multiple-arch dam, which the writer examined recently, nearly every buttress has developed a crack about $\frac{1}{8}$ in. in width. All these cracks begin a few feet from the arch barrel at about the middle of the height of the buttresses and extend through the buttresses to some distance above the foundation where they disappear. These cracks follow closely the line of the minimum inclined stresses in the buttresses. Measurements for more than a year show that the cracks decrease in width in summer when the temperature rises and widen somewhat when the temperature falls. No relation was found to exist between the width of the cracks and the height of

water in the reservoir. When the writer examined the dam, the reservoir was filled nearly to the crest of the spillway. If the water pressure had produced excessive shear or tension stresses, the movement would certainly not have stopped after opening the cracks in the unreinforced buttresses of this dam. Reinforcing the buttresses or suitably arranged inclined construction joints will prevent the occurrence of unsightly cracks, and such provisions should preferably be made in all future multiple-arch dams.

In addition to the dam described by the author, a number of multiple-arch dams on other than rock foundations have been built in recent years.

Cave Creek Dam, in Arizona, is about 120 ft. high and 1 690 ft. long, and is built on a gravel foundation. Some of the arches and buttresses of this dam extend 62 ft. below the ground surface. It is of interest to note that, notwithstanding the great depth of foundation of this dam, the total cost (approximately $575 000) was considerably less than the lowest bid on an earth-fill dam at the same site for the same reservoir capacity. The project drawings of the Cave Creek Dam have been shown in a discussion* by S. M. Cotten, Assoc. M. Am. Soc. C. E. It should be noted, however, that the maximum height of dam, as constructed, is about 120 ft. instead of 80 ft., as given on the original drawings shown by Mr. Cotten.

The Gillespie multiple-arch dam across the Gila River at Gila Bend, Ariz., is also founded on a sand and silt foundation. Similarly, as with the Sherman Island Dam, the buttresses of the Gillespie Dam are supported by a concrete foundation slab. A deep cut-off wall along the up-stream end of the slab and steel sheet-piling have proved to be satisfactory for preventing dangerous percolation under the dam. The height of this dam, however, is less than 20 ft. The bed-rock to which the steel sheet-piling was driven, is 40 to 50 ft. below the river bed.

A. Streiff,† Assoc. M. Am. Soc. C. E. (by letter).—The design of the Sherman Island Dam is interesting from many points of view. Large dams on soil foundations are plentiful in regions where the rivers traverse Pleistocene deposits of great thickness, as in India and the Middle Western United States. Few examples are to be found in Europe and, in the Eastern United States, rock foundations prevail. Perhaps the largest dams on soil foundations are in India and Egypt, measured by the flood discharge which passes over them, but as a rule the head on such dams is unimportant. In the Middle West there are a great many dams on soil foundations, and, in Canada, the Bassano Dam on the Bow River is a notable example.

The quotable examples, with heads of 60 ft. or more, combined with flood discharges of 85 000 sec-ft., are rare, and the designers and builders of the Sherman Island Dam are to be congratulated on the successful completion of a power dam of unusual dimensions on a soil foundation.

A head of 66 ft. ranks among the highest for such structures, if earth dams are excluded. To the writer's knowledge there are only a few others in the same class, the Avignonnet Dam, in France, on compact gravel, and the Junc-

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 402, Fig. 29.

† Cons. Engr., Jackson, Mich.

tion and Hodenpyl Dams, in Michigan, with heads of 55 and 67 ft., respectively, on soil foundations of variable composition.

It is of interest, therefore, to examine the design of the Sherman Island Dam. The percolation ratio, $\frac{\text{percolation distance}}{\text{head}}$, evidently did not cause much concern. A ratio of less than 4 was selected, which seems to confirm the fact that the larger ratios, as advocated by Bligh,* are not generally followed in the United States. Probably the shortest percolation distances have been used by Gardner S. Williams, M. Am. Soc. C. E., as, for example, in the Barton Dam on the Huron River in Michigan. The writer hesitates to use a factor of less than 4, although several structures with smaller ratios have been "in good standing" for years. The Croton Dam, in Michigan, with a head of 40 ft., built in 1907, and the Twin Branch Dam, on the St. Joseph River, in Indiana, with a head of 20 ft. and a percolation distance so short as to cause apprehension, but standing safely for twenty years, are other examples that may be mentioned.

There always remains the danger of shrinkage and "roofing" giving rise to piping, and sheet-piling is not always a secure protection. The writer at one time repaired a large leak in a lock on the Saramacca Canal in Guiana that had been caused by piping through four rows of sheet-piling. Another failure occurred several years ago at the Berrien Springs Dam, on the St. Joseph River in Michigan, after fifteen years of successful operation. A sudden leak in the tail-race, due to piping through a row of sheet-piling, was discovered just in time to save the structure.

From the author's description it seems that the two rows of sheet-piling in the Sherman Island Dam were exceptionally tight. The quantity of percolation is at wide variance with all the writer's previous experience. Such an effective reduction of the percolation was only found in the Wissota Dam (60-ft. head), but this was due to sealing the core-wall into a rock trench. In all the writer's observations of dams that rest on soil foundations, a greater percolation was found. At the Junction Dam, on the Manistee River in Michigan (55-ft. head), the earth-fill, composed of fine sand, was provided with a reinforced concrete core-wall resting on steel sheet-piling 40 to 50 ft. long, the full length of the fill. Upon raising the pond level, the percolation line on the down-stream side of the core-wall rose steadily within 10 to 15 ft. of, and simultaneously with, the pond level. Pipes arranged under the power-house showed that there was an irregular slope, which varied with the pond level, although the whole power-house rests on a bed of clay, varying from soft to hard, surrounded on all sides by steel sheet-piling at least 40 ft. long, and having a heavy blanket fill of fine sand on the up-stream side. The bearing pressures were as high as 4 tons per sq. ft. on the down-stream toe where the clay was hard. In this instance, it was conclusively demonstrated that a clay foundation does not necessarily prevent penetration of water pressure between it and the concrete structure it supports. At the Mio Dam, on the Ausable

* "The Practical Design of Irrigation Works", by W. C. Bligh, 1910.

River, a structure with a head of 28 ft. on a sand foundation, pipes arranged in the foundation slab of the power-house also showed a continuous percolation line which varied with the pond level. In the dams mentioned, the sheet-piling causes a drop in the line of percolation, but not nearly so great as that observed at the Sherman Island Dam.

Perhaps the arrangement of the test pipes has something to do with the observed results. In the dams cited, the arrangement of the pipes differed from those in the Sherman Island Dam; they terminated on the under side of the base slab, and were surrounded by a small pocket of gravel, laid there before the concrete was placed. It is well known that although percolation through soil may be small, on the plane of junction between earth and masonry, or steel, a cleavage due to shrinkage or unequal settlement, or due even to the act of cutting the adhesion of the earth as, for instance, in driving piling, may be expected. At the Junction Dam, the cleavage of the soil due to driving sheet-piling around the power-house released at several places some Artesian pressure which followed the piling and was, in many places, sufficient to erode the earth adjoining the pile and enlarge the hole. Such observations are very old; thus in the designs of brick retaining walls for ship-canal locks in Holland, it has always been considered good practice not to use steps to vary the thickness of the walls, but to taper the walls instead. Fig. 27 shows the Ceresco Dam, in Michigan, a small structure with a head of 15 ft. Holes drilled through the apron slab furnished gushers. Such a flow would have been impossible through the foundation material, and originated in the cleavage plane under, and adjacent to, the apron slab. It seems that the results found at the Sherman Island Dam for the magnitude of the upward pressure are not wholly conclusive and may be regarded as exceptional, and it will be prudent to apply them with reserve.

With regard to the foundation area to be assumed in calculating uplift, the writer prefers to be conservative. Such experiences as those referred to with the Junction Power-house, on a clay foundation, tend to promote this attitude. The field is rather unknown, although many investigations have been made. Such tests as those described by Mr. Meem do not seem wholly conclusive, and are open to various interpretations. Careful tests along that line were made in 1886 by Brennecke, an authority on foundations at that time.* He found the reduction in area practically nil for a grain size larger than 0.1 mm., and increasing sharply for smaller sizes. Although he gives formulas for practical use, his tests also seem subject to different interpretations and are of doubtful value for permanent structures.

The writer would like to inquire why a separate spillway was made for which a large rock excavation was necessary? The main dam and wing-wall are 725 ft. long, which would seem to be sufficient without the addition of another spillway 864 ft. long with the expense of a large rock excavation.

Where the spillway must be built on a soil foundation, there is no choice, and experience has shown that it can be done safely and at no extraordinary cost. Practice on this point differs, however; in many structures the soil

* "Grundbau", by L. Brennecke, Berlin, 1887, p. 125.

FIG. 27.—VIEW OF CERESCO DAM, IN MICHIGAN.

FIG. 28.—DAM AND POWER-HOUSE, CROTON, MICH., DURING FLOOD.

FIG. 29.—JUNCTION DAM, MANISTEE RIVER, MICHIGAN, DURING FLOOD.

foundation is merely protected by an apron of sufficient length, which practice requires long aprons, as the water acquires a horizontal velocity in sliding down the ogee and its maximum turbulence takes place at a considerable distance down stream from the dam.

British engineers long ago began the practice of making a tumble-bay, as in the Nira Weir, in India,* and concentrating the turbulence nearer the dam by a vertical drop in a basin formed by a counter weir, thereby diminishing the length of the apron. In Michigan, the Webber, Croton, Big Rapids, Foote, Mio, Cooke, Loud, Five Channels, and Junction Dams have been built, embodying that principle, on soil foundations, and have been satisfactory. The heads range from 26 to 55 ft. In the latest designs—the Alcona Dam (40-ft. head) and the Hodenpyl Dam (67-ft. head)—W. W. Tefft, M. Am. Soc. C. E., has cleverly replaced the special tumble-bay by that already available formed by the tail-race, and thereby effected considerable saving. In such structures, a discharge of 180 sec-ft. per lin. ft. of spillway has been found to be quite permissible and satisfactory.

Figs. 28 and 29 show the Croton and Junction Dams during flood. With these designs the soil foundation remains intact, whereas there have been many failures of rock foundations used as apron. Among the well-known examples are the erosions at McCall Ferry, Pa., and Keokuk, Iowa. After the flood of 1923 in New England, the writer noted such erosions at the Skowhegan Dam and the Ellsworth Dam, in Maine, where the rock is hard mica schist. The late James Dix Schuyler, M. Am. Soc. C. E., described several instances of erosion in his well-known work on dams.† In Switzerland, important rock erosion and undermining has occurred in many dams with large spillway gates, as at Rheinfelden, Laufenburg, Chevres, Hagneck, and several other places,‡ necessitating extensive repairs. In this country, the Holyoke Dam is a classical example. The old timber crib developed a large hole in the rock apron, and, filled with an extension crib, the process was repeated, until replaced by the present masonry dam with upturned toe, probably to transfer the erosion farther down stream.

Preference for rock aprons is, therefore, hardly sustained by experience, although the consequences of failure are not likely to be as destructive as in a structure placed on soil. However, the problem of design on soil may be regarded as solved, and no potent reason for avoiding spillways on soil foundations may be said to exist, except comparison of cost.

A further characteristic of the design of the Sherman Island Dam is the adherence to the prevailing practice in the East and South of using rollway and flash-boards. In the West and Middle West many more gate-controlled spillways are found, and the reason—possibility of maintaining highest pond level at all times—is of importance. One finds movable gate-controlled spillways almost exclusively in Europe. In the Middle West, where the Taintor gate was originated, gate control is often found, although other gate designs,

* "Irrigation in India and Egypt", by Buckley.

† "Reservoirs for Irrigation, Water Power and Domestic Water Supply", 1908.

‡ *Schweizerische Bauzeitung*, Roth, 1920.

such as rolling dams, are also used. The writer arranged an automatic spillway for 110 000 sec-ft. in the Wissota Dam. This preponderance in the use of gate control is due also to the frequent combination with earth dams, allowing less free-board and less fluctuation of head against the earth dam. These advantages disappear in the East, where earth dams are less numerous, and where water rights also permit an unusual rise not allowable permanently. The adoption of a fixed dam crest as an index to flowage right, a survival of old saw-mill days, is a distinct disadvantage for the intensive utilization of water power, and its modification in accordance with the requirements of sound engineering is desirable.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).—The writer appreciates the discussion which his paper has elicited, especially the fact that the interesting features received so much attention, namely, the uplift pressure under the main dam, the loss of head due to sheet-piling, and the flat up-stream slope of the arch barrels permitting the water to add weight for stability.

The writer is now conducting large scale experiments at the Rensselaer Polytechnic Institute Laboratories, with the co-operation of T. R. Lawson, M. Am. Soc. C. E., to determine the "effective" area of a foundation footing embedded in pervious soils of varying character subject to hydraulic uplift. It is hoped that these experiments will determine whether the total area of a foundation footing is reduced by soil contact, thus leaving only a portion to be acted upon by hydraulic uplift. When these tests have been completed the writer will offer the information to the profession through another contribution to the Society.

The International Paper Company has not made surveys at regular intervals since the development was put in commercial operation, and such surveys as have been reported to the writer are difficult to correlate with those previously made. It is to be regretted that the information given in Tables 2 and 3, could not be continued. From such information as has been received since the paper was prepared, the subsequent movement of the main dam is given in Table 4. The settlement shown in Column (2) reflects some of the settlement mentioned in Table 2, caused by raising the water level in the pond above the dam and by placing some of the sand filling under the arches. It must be remembered that Table 4 shows movement as surveyed along the walk on top of the dam, while the progressive settlement shown in Table 2 was based on movement as surveyed near the base of the dam.

Owing to the difficulties of surveying on top of the dam due to wind, heat waves, etc., the probable error in Table 4 is ± 0.06 in. Also, temperature changes will affect the position of the survey points on top of the dam, as expansion of the buttresses would tend to throw the points up stream.

Readings of uplift head in pipes under the main dam were not taken between October, 1924, and April, 1925. In April, 1925, four readings were made, one in June, one in July, and one in August, and Fig. 19 has been extended in Fig. 30, to show all observations of mean uplift pressure head and average water temperature since the records were commenced.

* Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., New York, N. Y.

TABLE 4.—Movement of Main Dam as Surveyed along Walk on Top.

Buttresses.	Settlement, in Inches:		Lateral movement, in inches, from April 29, 1924, to August 4, 1925.
	From April 19, 1923, to May 3, 1924.	From May 3, 1924, to August 5, 1925.	
(1)	(2)	(3)	(4)
3	+0.01	−0.01	+0.07
4	+0.07	0.0	−0.01
5	+0.05	+0.02	+0.13
6	+0.05	+0.02	+0.10
7	−0.08	−0.06	+0.10
8	−0.11	−0.02	+0.05
9	−0.22	−0.04	+0.22
10	−0.18	−0.05	+0.17
11	−0.25	−0.01	+0.12
12	−0.32	0.0	+0.23
13	−0.44	+0.04	+0.12
14	−0.59	−0.05	+0.25
15	−0.55	−0.05	+0.26
16	−0.47	−0.12	0.0
17	−0.43	−0.13	+0.32
18	−0.41	−0.10	−0.40
19	−0.50	−0.10	−0.07
20	−0.53	−0.07	−0.06
21	−0.60	−0.08	−0.04
22	−0.60	−0.04	+0.18
23	−0.56	−0.07	0.0
24	−0.48	−0.02	+0.01
25	−0.32	−0.06	−0.06
26	−0.35	+0.01	−0.02
27	−0.10	+0.04	−0.01
28	−0.07	0.0	−0.01
29	0.0	+0.01	+0.01
30	0.0	0.0	−0.06

Referring to Mr. Merriman's discussion, markings were not found on the topography, at or near the site of the main dam, which indicated an unknown flood in excess of that of March, 1913. There were ice scratches on the outcropping rocks, but these scratches were made in some pre-historic glacial period and could not be considered as defining modern flood levels. Records of the Hudson River have been kept by the State of New York and by the U. S. Geological Survey for thirty years, and by private companies for a longer period. The flood of 1913 is the greatest on record. Fig. 3 gives all the rock elevations known at the site. Borings made toward the middle of the river did not reach rock at a depth of 80 ft. below the river bed. If the rock slopes continue downward, as shown in Fig. 3, the depth of the gorge at the site might be about 190 ft. below the original river bottom.

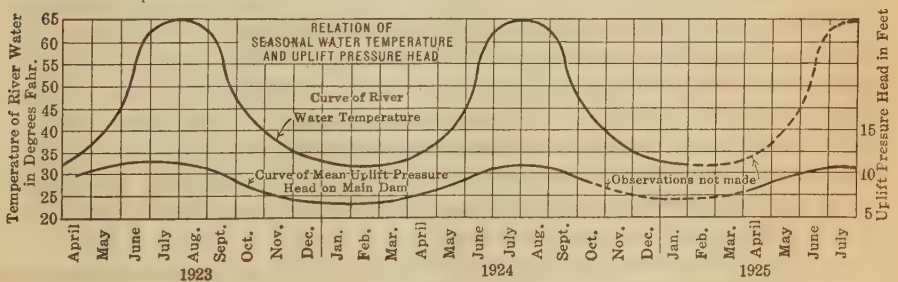

Fig. 30.

The Spier Falls Dam, also designed by the writer, located about 4½ miles up stream from Sherman Island, has a gravity section founded on rock, and the gorge at this site is shown in Fig. 31. The river valley at Spier Falls is more than twice as wide as that at Sherman Island, although the rock gorge below river level is much less in depth at the former than at the latter site. The rock gorge at Sherman Island, no doubt, has characteristics similar to that at Spier Falls.

The major portion of the base slab under Buttress 6, was not founded on rock, although the small scale of Fig. 3 might lead to that conclusion. The rock slope shown is an approximate average of the variations in sections across the site. The settlement of 0.40 in., given in Table 2, was, no doubt, a settlement due to compression of the more or less thin layer of sand between the base slab and rock surface.

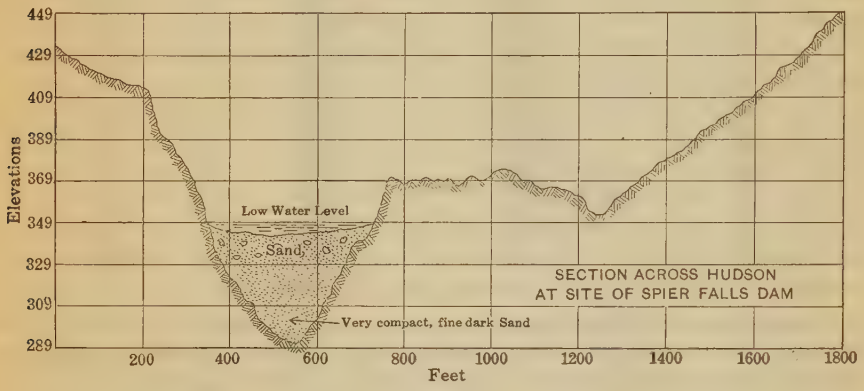

FIG. 31.

The uplift pressure under the dam (Fig. 18) and its seasonal variation (Fig. 19) are the plattings of observations made. The curves of uplift (Fig. 19) show the heights of the water in the test pipes above tail-water at the time the observations were made. Observations of heights of pond surface and of tail-water were made simultaneously with readings in the pipes. Variations in pond surface and tail-water did not effect the seasonal rise and fall of the water in the pipes. Temperature variations undoubtedly affect the voids in the sand. The viscosity of water also is affected by temperature. As temperature rises, the sand voids decrease, tending to resist the flow of water, while the water viscosity becomes less and probably more than offsets the reduction in flow area due to expansion of the sand. Within the temperature range, the reduction in porosity of the sand, due to expansion, is very much smaller than the percentage of decrease in viscosity of the water. While there may be an increase in velocity of water flowing through the sand due to a rise in temperature, the decrease in porosity is such that the actual change in the quantity of water flowing under the dam is probably not great from one temperature season to another.

In reply to Mr. Goldmark, the means taken to avoid cracking due to settlement was in the preparation of the rock bed to receive the concrete base of

the dam. In order that there should not be a sudden departure from solid rock to compressible sand, the rock was graded to leave a wedge-shaped sand bed. Grooves cut in the rock to receive the cut-off ribs were graded in a similar manner from full depth to zero. A tapering sand bed was placed on the rock beneath the concrete base. Of course, the thickness of this sand bed increased from the bank toward the river. This procedure removed pronounced points of contraflexure and extended the places of flexure in the concrete base slab over a wider area. A few fine cracks, chiefly vertical, did develop in some of the buttresses during the first year. They were due probably to shrinkage and temperature changes.

Replying to Mr. Jakobsen, the base was first poured; and after it had hardened, the buttresses were poured to the construction joint shown in Fig. 8. The lower portions of the arches were then poured and allowed to harden. The buttresses then were extended to the top, and after they had hardened, the upper portions of the arches (above Elevation 337) were poured. The pouring of the buttresses and of the arches was not continuous across the whole dam but proceeded from the north bank to the south; that is, a number of the north buttresses were poured, and then the lower portions of the arches, then more buttresses were poured toward the south and the lower portions of their arches. As the construction worked to the south, the northerly buttresses were raised and the upper portions of their arches were poured. The hardening intervals allowed some settlement and shrinkage to take place, which was leveled up if necessary. The arches were poured singly, one after another, and opportunity was given for vertical settlement before they were sprung.

The spillway was founded entirely on rock, the interstices of which were closed with concrete. It is hoped that uplift pressure does not exist beneath the spillway dam, as the rock was prepared with care.

Uplift pressure platted in Fig. 19 was the head above tail-water, so that Mr. Comstock's criticism does not apply. As stated in the paper, the uplift head was affected more by changes in tail-water than by those of pond water; and to allow for this, the writer arranged the ordinate of the curve to represent elevations above tail-water at the times of observation.

Ferritex was used on that portion of the dam built in the north coffer. It was omitted in the portion of the south coffer by order of the Paper Company, as time was pressing for closure of the dam. Only continued observation will tell whether or not this water-proofing was beneficial, but in the writer's opinion it should prove so.

In answer to Mr. Feld, the temperature readings of the water varied from day to day throughout a month, and the temperature curve shown in Fig. 19 is drawn as a smooth curve through the irregular platting. The apparent difference of about a month between maximum temperature and maximum uplift on the curves is probably more or less accidental. As this is the first time any relationship between temperature and uplift has been disclosed by actual observations, the writer admits that other conditions, such as changes in head, piping, and silting of pond, may affect the uplift, in addition to temperature. A too close analysis, with the limited data at hand, might lead to an erroneous conclusion.

Replying to Mr. Henny, observations made at various places in the development indicate that the sand under the main dam varied from coarse near the top of the river bed to finer grains as the depth increased; and that the approximately horizontal layers were in part cross-bedded and in part lenticular in shape. Strata of "clayey sand" helped to create lenticular formations.

The writer would state in reply to Mr. Turner that *Water Supply Paper No. 200,* U. S. Geological Survey, gives a number of diagrams showing crests and nappes of the type used for the Sherman Island spillway, in which the discharges increase for the depth of water. As the flood depth on the crest will be twice that given in the diagrams, it would seem that a coefficient of more than 4 would be justified. A curved crest, like that at Sherman Island, is not as effective as a straight crest at right angles to the flow. Mr. Turner's diagrams (Figs. 25 and 26) do not represent the course of the water as it approached the crest, as at high floods the water surface will be near Contour 360 as shown in Fig. 2, and the width of the river will be much greater than Mr. Turner's diagrams would indicate. At the up-stream side of the horseshoe bend, the material was excavated so as to provide an approach channel, and during the floods of 1923 and 1924 the water passed the crest of the horseshoe bend approximately at right angles. There was some diagonal approach along the sides between Sections 3 to 9, and 17 to 20, as marked on Figs. 25 and 26, but the departure from the normal was not great. During the floods of 1923 and 1924, the discharges over the crest agreed very closely with the theoretical curve prepared for the design of the spillway dam. While these floods did not approach the maximum, there was no evidence that the dam would not discharge the water for which it was designed. The wasteway channel discharged the water practically uniformly over its whole width at a line opposite the end of the main dam.

The wasteway channel should be kept clear, because the higher velocities have a tendency to block the channel by lifting loosened stones from its bottom. The waste water appears to reach a maximum velocity beyond which increased discharge is accomplished only by increased depth. It is not expected that the spillway crest will become submerged as there is a steeper slope at the upper end of the wasteway channel than at points farther down stream. The writer agrees that, in times of excessive flood, a study of the discharge over this spillway and through its wasteway channel would be instructive.

The writer was interested in Mr. Streiff's discussion and his references will be helpful to many engineers. It is a difficult matter to compare dam designs unless one is fully acquainted with the physical and geological formations existing at the sites under comparison, as well as with other essential requirements governing the selection of design. It was felt safer to excavate the rock for the waste channel in order that the spillway dam could be founded on rock, and, furthermore, a large quantity of the rock was required for concrete aggregate. A spillway crest of more than 850 ft. was considered essential in order to prevent interference with tail-water elevations at Spier Falls Dam in times of flood.

MEMOIRS OF DECEASED MEMBERS

JOHN AUGUSTUS OCKERSON, Past-President, Am. Soc. C. E.*

DIED MARCH 22, 1924.

John Augustus Ockerson, the son of Jans and Rose Jans Datler Akerson, was born on March 4, 1848, at Slattarod, Province of Skane, Sweden.

When two years of age he was brought by his parents, with their other children and some relatives, to the United States. While journeying West in an emigrant wagon, both parents and the oldest son died of cholera just before reaching Chicago, Ill. The remainder of the family proceeded in the care of their relatives, who settled near Elmwood, Ill., where young Ockerson grew to manhood, attending the public schools and the Elmwood High School, finally matriculating in the State University.

In Sweden, the name is spelled Akerson, but on coming to America, the family expressed the Swedish pronunciation in English by spelling it Ockerson.

When only sixteen years of age, but passing for eighteen, young Ockerson was mustered into the United States Military Service for the Civil War, at Elmwood, on June 1, 1864, as a Private in the 132d Illinois Infantry, but was mustered out with his Company on October 17 of the same year. However, at Red Wing, Minn., on January 30, 1865, he re-enlisted, this time in the 1st Minnesota Heavy Artillery and served until September 27, when he was mustered out with his Battery.

In the fall of 1869, he entered the University of Illinois, then known as the Illinois Industrial University, and while pursuing his studies there, took an active part in student and university affairs. During the summer vacation of 1871, he was engaged as a Recorder on the survey of the Great Lakes (U. S. Lake Survey), and during the vacation of the following year was a Levelman on location for the Atchison, Topeka, and Santa Fé Railroad. He was graduated in 1873 from the course in Civil Engineering, receiving the degree of Bachelor of Science, and subsequently that of Civil Engineer. Thirty years later, his Alma Mater conferred on him the honorary degree of Doctor of Engineering. The title, "Colonel", frequently applied to him was only by courtesy, but was never pleasing to him.

From 1873 to 1878, Mr. Ockerson was again employed on the survey of the Great Lakes, with the title of Assistant Engineer, being engaged in topographic and hydrographic surveys, in triangulation, and the measurement of primary base lines. During this period he was for a time, in 1876, Government Inspector on the survey for the Eads jetties at the mouth of the Mississippi River.

In 1879, he was transferred to the newly created Mississippi River Commission, and, until 1887, was Principal Assistant Engineer in charge of surveys and physical examinations from the head-waters of the river to the Gulf of Mexico.

* Memoir prepared by the following Committee: I. O. Baker, J. W. Skelly, and William S. Mitchell, Members, Am. Soc. C. E.

In the latter year, Congressional appropriations for rivers and harbors having been greatly reduced, Mr. Ockerson left the service of the Commission, and was engaged, from 1888 to 1889, as Manager of a gold and silver mine in Colorado. In 1890, he returned to the Mississippi River Commission and served as its Principal Assistant Engineer until 1898, when, by appointment of President McKinley, he became a member of the Commission, in which position he remained for twenty-six years, until his death. He took an active part in its deliberations and in the direction of all work in its charge. During his later years of service, in recognition of his ability and thorough knowledge of the work of the Commission, he was usually Chairman of its leading committees, those on design and construction of levees, of hydraulic dredges and dredging, of bank protection, etc.

The Commission maps of the river, made by him and under his direction, are exceptionally full and accurate, and are in great demand both in this country and abroad.

In the later years of his life, Mr. Ockerson was active also as a Consulting Engineer, serving in advisory capacity on many important works, including the Chicago Drainage Canal, in 1907; Big Wood River Dam in Idaho, in 1909; East Side Levee and Sanitary District, East St. Louis, Ill., from 1908 to 1912; Little River Drainage District, Missouri, in 1914; and Conservancy District, Miami Valley, Dayton, Ohio, in 1914.

Perhaps his most outstanding individual work in engineering was the building of levees along the Colorado River in 1910, to hold back its flood waters which threatened to overflow into the Salton Sea. He was appointed by President Taft, late in the summer of 1910, to take charge of the work for which Congress had recently appropriated $1 000 000. The work was difficult and trying, owing to the light silt soil on which the levee was founded and to the necessity of haste to finish before the season of floods, and, further, to the fact that the work was to be done on the Mexican side of the International Boundary, with teams, tools, materials, etc., from the American side, to protect settlers and property in the United States, which introduced serious complications in the Customs regulations. The regulations of the United States Government concerning the expenditure of public funds so seriously hampered the work, in which expedition was vital, that, at times, it became necessary to telegraph to the Interior Department at Washington, to ask the State Department to request the Government of Mexico to authorize the Mexican Customs Officials to permit the entrance of emergency supplies, and then to await reply through the same circuitous route. In addition, the Indian inhabitants of the adjacent Mexican territory were in active insurrection, and the Republic of Mexico itself in the throes of revolution. For this work, Mr. Ockerson received the personal commendation of President Taft for his "pertinacity under the most trying circumstances."

At the Louisiana Purchase Exposition in St. Louis, from 1902 to 1905, Mr. Ockerson was Chief of the Department of Liberal Arts, and also was a member of the Superior Jury of Awards. He had been previously a member of the International Jury of Awards of the Paris Exposition in 1900.

His study of river improvements and related features took him to Europe on three Government investigations of waterways and seaports. He was an official delegate of the United States to four International Navigation Congresses.

At the St. Louis Exposition, and on his European trips, Mr. Ockerson received honors and decorations from many foreign Governments, among which were Officier de l'Instruction Publique and also Merité Agricole (France); Knight of the Crown (Italy); Knight and also Commander of the Order of Vasa (Sweden); Knight of the Crown (Germany); Knight of the Order of Leopold (Belgium); Officer of the Order of the Double Dragon, first grade (China); and a silver medal for special services, from Russia. Because of his official position on the Mississippi River Commission, under the law, he could not accept the insignia of these orders, and they were left, until after his death, in the custody of the State Department in Washington.

Mr. Ockerson was the author of numerous official technical reports from 1881 until his death. Many of his papers and discussions have been published in the *Transactions* of the Society, in the *Journal* of the Association of Engineering Societies, and in various other engineering periodicals. He contributed papers to the *Proceedings* of the International Congresses of Navigation at Paris in 1900; at Milan in 1905; at St. Petersburg in 1908; at Philadelphia in 1912; and to the Engineering Congress at Glasgow in 1901. He was intimately identified with early experiments in hydraulic dredging and his first and elaborate report on that method of opening and maintaining channels for navigation in rivers, has been a milestone in engineering history. His long association with, and personal knowledge of, the Mississippi River, as well as his many clear, forceful, and instructive writings, gave him an eminent position among civil engineers engaged in river regulation in both this country and in foreign countries.

He died at his home in St. Louis, and was laid to rest in Bellefontaine Cemetery in that city.

Mr. Ockerson was married on November 3, 1875, at Detroit, Mich., to Helen M. Chapin, who died in March, 1886. On June 4, 1890, he was married to Clara W. Shackelford, of St. Louis, Mo., who survives him.

He was twice Vice-President and twice President of the Engineers' Club of St. Louis, and was a member of the Grand Army of the Republic, of several social clubs, and of various civic organizations.

In person, Mr. Ockerson was tall and handsome, with a dignified and quiet grace of manner. He was affable, easily approached, and in public spoke readily with clear and forceful expression. He will continue to live in his writings, and especially in his labors for the advancement of engineers, and his life-long efforts for self-improvement will stand as an inspiration to worthy young Americans.

Mr. Ockerson was elected a Member of the American Society of Civil Engineers on June 7, 1880. He served as Vice-President of the Society in 1907 and 1908, and as President in 1912.

SIR MAURICE FITZMAURICE, Hon. M. Am. Soc. C. E.*

DIED NOVEMBER 17, 1924.

Maurice Fitzmaurice was born on May 11, 1861, in County Kerry, Ireland. He was educated at Trinity College, Dublin, obtaining the degrees of Bachelor of Arts and Bachelor of Engineering.

From 1883 to 1885, he was articled to the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., under whom, in conjunction with Sir John Fowler, he was employed between 1885 and 1888 on construction work on the Forth Bridge. From October, 1888, to June, 1891, he was in responsible charge of the construction of the Chignecto Ship Railway in Canada, and, subsequently, until May, 1892, he was engaged on the designs and estimates for replacing cast-iron bridges on the London, Brighton & South Coast Railway with steel structures.

Later in May, 1892, Mr. Fitzmaurice was appointed Resident Engineer under Sir A. R. Binnie (then Chief Engineer to the London County Council) during the construction of the Blackwall Tunnel.

In 1898, he was appointed Resident Engineer to the Egyptian Government on the construction of the Assuan Dam, the late Sir Benjamin Baker being the Consulting Engineer.

Shortly after his return to England in 1902, and following the retirement of Sir Alexander Binnie, Mr Fitzmaurice was appointed Engineer-in-Chief to the London County Council. The following are some of the works which were designed and carried out by him either entirely or in part, before he retired in 1912: The duplication of the main drainage of London; the London County Council tramways; the Rotherhithe Tunnel; the underground tramway from the Thames Embankment up Kingsway to the north; the Greenwich and Woolwich Footways under the Thames; the Woolwich Ferry jetties; the coal wharf for the tramway power station of the London County Council, at Greenwich; the new Vauxhall Bridge; the prolongation of the Thames Embankment on the north bank up stream from the Houses of Parliament; and the embankment for the new County Hall on the south bank of the Thames. It was in connection with the laying of the foundation stone of the new County Hall that he received the honor of Knighthood.

After resigning his position as Chief Engineer to the London County Council to engage in consulting work, Sir Maurice joined the firm of Messrs. Coode, Son & Matthews, afterward changed, on the retirement of Sir William Matthews, to Messrs. Coode, Fitzmaurice, Wilson & Mitchell, with which firm he remained to the time of his death.

Among the more important works with which he was associated during this period are: Harbor works at Peterhead, Scotland, Fishguard, Wales, Colombo, Ceylon, Singapore, Straits Settlement, and East and West Africa.

An undertaking to which he devoted much attention and for which his firm acted as Consulting Engineers was the Gezira Irrigation Scheme, which included the building of the Sennar Dam on the Blue Nile.

* Memoir prepared by Maurice F. G. Wilson, Esq., London, England.

In 1913, on behalf of his firm, Sir Maurice visited Australia with a view to giving advice concerning the construction of naval harbors and works; similarly, in 1920, he visited Hong Kong, China, in connection with proposed harbor improvements at that port. In 1912, he was appointed Chairman of the Advisory Committee of the Admiralty on Naval Works, which office he held until 1919.

Associated with Mr. Basil Mott, Sir Maurice was engaged on the following projects: Advising on the question of a further tunnel under the Mersey to connect Liverpool with Birkenhead; designing the new high-level bridge to be constructed at Newcastle-on-Tyne; and advising what was best to be done with Waterloo Bridge when it showed signs of failure.

During the World War, in addition to other activities, he served as Chairman of the War Office Committee on Civilian Labor on the London Defences and as a member of the War Office Committee on Hutted Camps. He twice visited the British Front on the Continent for the purpose of advising on questions of drainage.

During 1918-19 he served as Chairman of the Nile Projects Committee of the Foreign Office and from 1917 to 1919 as Chairman of the Canal Control Committee of the Board of Trade. In 1919, he was appointed Chairman of the Treasury Committee on Aerodrome Accounts. Sir Maurice was a member of the Royal Commission on Fire Prevention, of the Advisory Council of the Science Museum from 1915 to 1921, of the Advisory Council on Scientific and Industrial Research, and of the International Technical Commission, Suez Canal. He was also Colonel Commandant of the Engineer and Railway Staff Corps.

He received the Order of the Mejidieh, 2d Class, in 1901, for his work on the Assuan Dam, and was created a Companion of the Order of St. Michael and St. George the following year. He held the honorary degree of LL.D. from Birmingham University, and had been a Fellow of the Royal Society since 1919. He entered The Institution of Civil Engineers as a Student, became an Associate Member in 1887, was transferred to the grade of Member in 1893, and served as President in 1916. He received for contributions to the *Minutes of Proceedings* of The Institution the Telford and Watt Medals, a Telford Premium, and a Miller Prize. He was also a member of The Institution of Mechanical Engineers, an Honorary Fellow of the Society of Engineers, an Honorary Member of the Royal Engineers' Institution, and a member of the Engineering Institute of Canada.

He was married in 1911 to Ida, daughter of the late Col. Edward Dickinson, R. E., of West Lavington Hill, Midhurst, Sussex, who, with two daughters, survives him.

Sir Maurice was elected an Honorary Member of the American Society of Civil Engineers on June 19, 1922.

ALBERT HILANDS ACHER, M. Am. Soc. C. E.*

Died August 26, 1924.

Albert Hilands Acher, the son of Albert M. and Mary (Hilands) Acher, was born at Greenville, Pa., on June 6, 1885. Both his parents belonged to families locally well-known and respected; his maternal grandfather was one of the first to enlist for the Civil War. In his early childhood the family moved to Canada for a few years, returning to Pennsylvania when he was ten years old. He attended the public schools in Toronto, Ont., Canada, and Grove City, Pa.

In 1904, after one year at the Grove City College, Mr. Acher was nominated to a cadetship at the United States Military Academy by Representative Joseph R. Sibley, after a competitive examination, from a large number of candidates. He took high rank at West Point, both in his military duties and his scholastic work. As a Cadet Lieutenant in his first class year, he was placed in charge of the preliminary training of the entering class of new cadets—a responsible appointment and clear proof of the confidence placed in him by his superior officers. He was graduated in 1909 and commissioned a Second Lieutenant in the Corps of Engineers, U. S. Army.

After short tours of instruction on the Isthmus of Panama and in important Engineer Districts in the United States, Lieutenant Acher served as student at the Engineer School of Application and with the Battalion of Engineers at Washington Barracks, D. C., until the spring of 1912, when he was assigned to duty on the Canal Zone under the Isthmian Canal Commission. He served for a few months as Junior Engineer in the Light-House Sub-Division under Mr. W. F. Beyer and was then transferred as Assistant Engineer to the Fortification Division, of which he was in charge during his last nine months on the Isthmus. In July, 1915, he returned to the United States and served for two years as Assistant in the U. S. Engineer District with headquarters at Los Angeles, Calif., his principal duty being the planning and building of seacoast batteries at Fort Rosecrans and San Pedro, Calif. He then served for a short time as Adjutant of the 4th Regiment of Engineers. This was followed by six months' duty in charge of the Seattle, Wash., Engineer District, from which he was relieved in December, 1917, and sent again to duty with the 4th Regiment of Engineers, with a temporary commission as Major. As the regiment was preparing for overseas service in the near future, this change gave him great satisfaction.

The regiment was assigned to the 4th Division and went to France in the spring of 1918, arriving in Bordeaux on May 12. Major Acher was promoted to the rank of Lieutenant-Colonel and Colonel of Engineers in the temporary forces on February 14 and August 1, 1918, respectively. In command of his regiment and as Division Engineer of the 4th Division, he took part in the Second Battle of the Marne, July 18 to 23; in the Vesle River Campaign, August 3 to 12; and in the Meuse-Argonne Offensive, Sep-

* Memoir prepared by Maj.-Gen. H. F. Hodges, U. S. A. (*Retired*), M. Am. Soc. C. E.

tember 26 to October 10, 1918. He was then transferred to the 102d Engineers as Commander and also assigned as Division Engineer of the 27th Division; in these capacities he took part in the operations of the Second American Army Corps with the British Fourth Army under General Sir Henry Rawlinson until the Armistice on November 11, 1918. Those operations included forcing La Selle River and driving the enemy back to the line of the Sambre Canal.

After the Armistice, Colonel Acher was detached from the 27th Division before its return to the United States and was assigned as Chief of the Italian Division of the War Damage Section of the American Commission to Negotiate Peace. On the conclusion of this duty, he closed up the affairs of the Light Railways of the American Expeditionary Forces, which occupied him until his return to the United States in June, 1919. He was then placed on duty in the Office of the Chief of Engineers, U. S. A., in Washington, D. C., where he remained until he resigned from the Army on November 6, 1919, with the rank of Colonel in the temporary forces and Captain in the Regular Army.

During his service in the Army, Colonel Acher earned in all his duties the commendation of his superior officers, and the admiration and respect of those who worked under him. Maj.-Gen. George W. Goethals, U. S. A. (*Retired*), M. Am. Soc. C. E., said of his work on the Isthmus:

"It soon became apparent that, combined with a pleasing personality, he [Colonel Acher] possessed unusual ability and those rare and unmistakable qualities that constitute a leader. Were I to name certain of these that, to me, stood out above others in this connection I should mention his intelligence, fairness, frankness, loyalty, willingness to assume and carry responsibility, and his ability to handle men. I can speak with authority and feeling of his work on the Canal for I had many opportunities to observe it first hand and it never failed to measure up to the highest standard. I always found him very energetic, bright, quick in thought and action, possessing good judgment and good hard common sense; firm, yet fair in all his dealings; conscientious, painstaking; kind, generous and thoughtful of others; and, above all, intensely human. These same qualities I have heard mentioned by those having knowledge of his services in France during the World War, and they account for the success which he subsequently attained in civil life. He won and held the confidence, respect, and admiration of all with whom he came in contact."

Colonel Acher was highly commended by Maj.-Gen. W. L. Sibert, U. S. A. (*Retired*), M. Am. Soc. C. E., then in command of the South Pacific Coast Artillery District, for the economy and speed with which he had built certain important seacoast batteries. The Commander of the 4th Division, A. E. F., in recommending Colonel Acher for the Distinguished Service Medal, wrote that:

"During the operations on the Vesle River, the 4th Engineers, under the command of Colonel Acher, and inspired by his personal example, distinguished itself by building numerous bridges across that river under heavy artillery, machine gun, and rifle fire, and building other bridges as soon as these were destroyed. During the Meuse-Argonne Campaign, Colonel Acher planned, supervised and completed the construction of the road from the Town of Esnes to Malancourt, a task of such vital importance to the success

of the American forces that without it the campaign would inevitably have been brought to a halt. This work involved the construction of a new road over the base of what four years before had been a road, but which had been torn to pieces by the shell fire of both the Germans and the French. Work was started at 9:30 P. M. and accomplished at 2 P. M. on the following afternoon. Between these hours 2 miles of road was built by four companies of Engineers, 40 000 sand bags being used. The road later became one of the two main arteries of transport of two Army Corps. It was due entirely to Colonel Acher's foresight in planning the amount and character of supplies necessary and arranging the order and distribution of his forces that this road was built and the bridges over the Ruisseau de Forges were completed in ample time for the passage of the artillery and trains.

"Colonel Acher is a natural leader of men and has implanted in the 4th Engineers a spirit of helpfulness and an *esprit de corps* that makes the organization not only a most efficient technical unit, but a highly trained combat unit as well. During the three major operations in which the regiment has participated, Colonel Acher has, regardless of his own safety, exposed himself to hostile fire at all points where organizations of his regiment were working, and, by his personal courage, has been an inspiration to his men."

Another of the Commanders of the 4th Division, himself an ex-Infantryman, stated, in reporting on Colonel Acher's efficiency, that:

"I consider him the best Engineer Officer I have ever known. I feel also that he would make an excellent infantry commander in war.

"By his good judgment, energy, and zeal, he made the advance of the 4th Division possible in the Meuse-Argonne offensive. He holds a peculiar control and influence over officers and soldiers under him, which increases manyfold their efficiency."

Another of the Commanders of the 4th Division writes:

"Acher took command of the regiment before its first action under fire. The Division owed him much. Not only was he an excellent executive and administrator, but he had the faculty of looking ahead, so that when movements were projected, it was usually found that he had already begun preparations.

"His loyalty and spirit of service I have never seen excelled. His strongest characteristic was directness in thought and speech. Quick to apprehend, he made prompt, direct answers to questions and equally prompt, sound, practical decisions. To hesitate, to consider trifles, or to move in anything but a straight line to his object was wholly foreign to his nature. Personally, we were all devoted to him."

The Commanding General of the 27th Division writes that the 102d Engineers, under Colonel Acher's command, did exceptionally fine work in the assault on La Selle River and the heights beyond, and that,

"Colonel Acher combined with a splendid technical and professional preparation for regimental command a disposition and human attitude toward his men which soon won their confidence and affection."

In a formal letter to the American Corps Commander the Commanding General of the British Fourth Army expressed his thanks for and his appreciation of the excellent work done by the 102d and 105th Regiments of Engineers, under the command of Colonel Acher and Colonel J. H. Pratt, respectively.

For his "exceptionally meritorious and distinguished services" as commanding officer of the 4th Engineers and, later, of the 102d Engineers, Colonel Acher was awarded the Distinguished Service Medal by the Commander-in-Chief of the A. E. F., in the name of the President.

After his resignation from the Army, Colonel Acher accepted and held for a year, the position of Chief Engineer of the U. S. Food Products Corporation. He then associated himself with G. S. Bergendahl, M. Am. Soc. C. E., in the formation of The Bergendahl and Acher Company, an engineering and contracting company which specialized in the construction of forms for reinforced concrete. In December, 1922, Mr. Bergendahl severed his connection with the Company, which continued, under changed style, however, as The B. and A. Company, with Colonel Acher as President. Its operations were very successful. It built forms for various reinforced concrete buildings in many cities, for example, the American Furniture Mart in Chicago, Ill., one of the largest buildings in the world, measured by cubic contents.

Although separated from the Regular Army, Colonel Acher recognized always his obligation to do his part in the defense of the country when needed, and at the time of his death held a commission as Colonel in the Officers' Reserve Corps.

He was busily engaged on the operations of his company and on plans which he had almost perfected for extending such operations materially, when he was taken ill in the early summer of 1924. The illness, at first apparently trivial, developed serious features, and terminated fatally on August 26, 1924.

Colonel Acher was married in 1913 to Frances, daughter of Gen. and Mrs. H. F. Hodges. His widow and three daughters survive him.

He was a man of great energy and force of character, straight-forward and direct in his dealings, holding fast to his own purpose, but uniformly fair toward his competitors as well as his associates. The men employed in his work were sure of just and considerate treatment at his hands and all looked on him with admiration and respect, tinged with affection. They felt his death as a personal loss. He was a capable and successful engineer, a brave and distinguished soldier. His early death ended a career of unusual achievement and still more unusual promise.

Colonel Acher was elected an Associate Member of the American Society of Civil Engineers on April 7, 1915, and a Member on November 25, 1919.

GEORGE GRAY ANDERSON, M. Am. Soc. C. E.*

DIED DECEMBER 23, 1923.

George Gray Anderson was born in Aberdeen, Scotland, on April 20, 1858. He received his early education at the Robert Gordon's College in his native city. During the years 1872-74, he was a student at the University of Aberdeen, where he won special honors in mathematics and physics. From April, 1874, to April, 1879, he served as an articled pupil with Walker and Beattie, Engineers, of Aberdeen. On the termination of his apprenticeship, Mr. Anderson became an Assistant Engineer on the London-Rugby Division of the of the London and Northwestern Railway in England.

In April, 1880, Mr. Anderson came to the United States and settled in Denver, Colo., where he became Assistant Engineer in the employ of the English company then promoting the Northern Colorado Irrigation Company's project to serve lands adjoining Denver. In June, 1883, he was appointed Chief Engineer and General Superintendent of the Platte Land Company of Denver, which carried out the Northern Colorado Irrigation Company's project and constructed the Larimer and Weld, Loveland and Greeley, and Platte Valley Canals. These projects embraced in all approximately 200 miles of main canals.

In June, 1890, Mr. Anderson severed his connection with the Platte Land Company to become a partner in the prominent consulting engineering firm of Campbell and Greene, of Denver, the new firm taking the name of Campbell, Greene and Anderson. In 1891, on the retirement of Mr. Greene, the firm became Campbell and Anderson and continued under this name until 1896 when the partnership was dissolved.

From 1896 to 1916 Mr. Anderson practiced as a Consulting Engineer with offices in Denver. In 1916, on account of his wife's health, he moved to Los Angeles, Calif., where he continued his consulting practice until his death.

His recognized ability brought him an extensive practice throughout the Western States and in Canada. Among his numerous professional engagements, the following more important ones may be mentioned:

1896-1916, Expert on rates, quality, and pressures, Denver Union Water Company; construction of the Irrigation System of the Alberta Railway and Irrigation Company, Alberta, Canada; investigation and report on Canadian Pacific Irrigation System, Alberta; report on fountain water supply for Pueblo, Colo.; construction of water-works for the City of Greeley, and of Clear Creek Reservoir, Colorado; report on Sacramento Valley Irrigation District, California; appraisal of water rights of Denver Union Water Company; study and report for Dominion of Canada on international issue in St. Mary's River, Alberta *vs.* Montana; Expert in the case of Twin Lakes Placer Company *vs.* City of Pueblo; reports on Laramie Valley Irrigation District, Wyoming, and on Southern Alberta Land and Irrigation Project, Canada; construction of Schaefer Dam and Sanchez Reservoir, Colorado; recon-

* Memoir prepared by the following Committee: Harry Hawgood, *Chairman*, Lyman E. Bishop, F. G. Dessery, and John C. Ulrich, Members, Am. Soc. C. E.

struction of the Otero Canal System and report on the La Plata Land and Canal Company's System, Colorado; examination and report on Buffalo Basin, Riverside and Prewitt Reservoirs, and on a gravity water supply for Grand Junction, Colo.; investigation of water supply of Spring Valley Water Company, San Francisco, Calif.; study of water rights of hydro-electric properties of Pacific Gas and Electric Company, San Francisco; and appraisal in conjunction with Leonard Metcalf, M. Am. Soc. C. E., of properties of Denver Union Water Company. During this period Mr. Anderson also served as a member of the Board of Consulting Engineers of the Imperial Irrigation District of California.

1917-1923, Investigation and report on power possibilities of the water supply of Colorado Springs, Colo., and for the Government of Alberta on the Lethbridge Northern Irrigation District, and on various projected irrigation districts in Alberta, Canada; and an examination and report on the failure, during the Pueblo flood in June, 1921, of Schaefer Dam, Colorado.

In 1918, Mr. Anderson had a prolonged attack of influenza and in 1923 suffered a second attack, to the serious complications of which he ultimately succumbed.

Interested in irrigation, Mr. Anderson contributed papers to the *Transactions* of the Society on "Irrigation in Colorado"* and, on "The Effect of Alkali on Concrete"†. The Colorado Scientific Society has also published a paper sented a paper on "Irrigation in Colorado" at the Sixteenth National Irriga- by him on "Some Aspects of Irrigation Development in Colorado"‡. He pre- tion Congress, 1908, and another on "The Combination of Water Resources for Irrigation and Hydro-Electric Purposes," at the Second Pan-American Scientific Congress.

Mr. Anderson was a man of strong individuality, unlimited energy, and tireless industry. During his long residence in Colorado he earned and enjoyed a prestige in his profession unexcelled by that of any confrere. His charming personality gathered to him a host of friends, among whom were some of the most prominent citizens of Colorado, and, in later years, of California, to whom the news of his untimely and unexpected death came as a distressing shock and by whom he will always be remembered with affection and respect.

The younger members of the profession found in Mr. Anderson an approachable, kindly man ever ready to give them counsel in their work or aid them in obtaining employment.

The seriousness with which he accepted his engagements and discharged his responsibilities to the profession is illustrated by his insistence in October, 1923, only two months before his death, in rising from a sick bed in Los Angeles and crossing the Continent, accompanied by his wife, to attend the Fall Meeting of the Society at Richmond, Va. He was able to attend, in a wheel-chair, one meeting of the Board of Direction and thereafter for the remainder of his stay in Richmond was unable to leave his bed. Undoubtedly, this journey, in his weakened condition, hastened his death.

* *Transactions*, Am. Soc. C. E., Vol. LXII (1909), p. 1.

† *Loc. cit.*, Vol. LXVII (1910), p. 572.

‡ *Proceedings*, Colorado Scientific Society, Vol. IX (1908–10), p. 273.

Mr. Anderson was married at Denver, on October 31, 1883, to S. Ella Beck, daughter of Isaac T. Beck, of Denver. He is survived by his wife, two daughters, and a son, Robert Anderson, who is serving in the Flight Research Department of the United States Aviation Service.

He was a member of the Institution of Civil Engineers, the Engineering Institute of Canada, the Colorado Scientific Society, and the Seismological Society of America, and an Honorary Member of Tau Beta Pi. He was also a member of the Denver and Mile High Clubs of Denver, and of the University and Celtic Clubs of Los Angeles.

Mr. Anderson was elected a Member of the American Society of Civil Engineers on February 7, 1906, and, at the time of his death, was a member of the Board of Direction. He took a keen and active interest in the Society's affairs and will long be remembered for his forceful work on the revision of the Constitution as amended by the Board of Direction on March 1, 1922. He was also a Past-President of the Colorado and Los Angeles Sections of the Society.

JOHN CUMMINGS AUTEN, M. Am. Soc. C. E.*

DIED OCTOBER 15, 1923.

John Cummings Auten, the son of Wesley and Mary Nelson Auten, was born at Montandon, Pa., on August 31, 1870. He was of Huguenot descent and spent much time and labor collecting the history of his family.

In August, 1887, Mr. Auten entered the service of the Pennsylvania Railroad and remained with that Company all his life. He filled, successively and successfully, the following positions: August 5, 1887, Rodman in the Engineering Corps of the Sunbury Division; May 1, 1894, Assistant Supervisor on the Sunbury Division; November 1, 1894, promoted to the position of Supervisor of that Division; October 1, 1898, transferred to the Baltimore Division and, on December 10, 1901, to the Middle Division. On April 10, 1902, he was promoted to the position of Assistant Engineer of the Elmira and Canandaigua Division and, on June 1, 1903, was transferred to the West Penn Division where he served until April 1, 1907, when he was transferred to the Eastern Pennsylvania Division. On November 16, 1908, he was transferred to the West Jersey and Seashore Railroad and on January 15, 1910, was appointed Division Engineer of the Maryland Division. On February 11, 1914, Mr. Auten was promoted to the position of Principal Assistant Engineer of the Philadelphia, Baltimore and Washington Railroad, in which position he served until December 16, 1919, when he was transferred to the office of the General Superintendent of the Southern Division at Wilmington, Del., and on May 1, 1920, to the Baltimore Division, to which Division he was attached at the time of his death.

Mr. Auten was always fair and considerate with those under him and his genial disposition made him a favorite with his associates. He was honored and esteemed by his superior officers. In private life, he was a devoted husband and father.

On April 23, 1916, Mr. Auten was installed and ordained an Elder of Westminster Presbyterian Church, Wilmington, which honorable position he held until his death. He was an enthusiastic and devoted member of the Order of Masons, being a member of the Blue Lodge, the Scottish Rite, and St. John's Commandery of Wilmington, and a member of the Shrine, Lulu Temple, Philadelphia, Pa. He was also a member of the City Club and the Whist Club of Wilmington.

On October 14, 1904, Mr. Auten was married to Ella Hawkins, of Parkton, Md., who, with two children, John Hawkins and Elizabeth, survives him.

Mr. Auten was elected a Member of the American Society of Civil Engineers on May 13, 1918, and took great interest in its activities.

* Memoir prepared by Franklin Duane, Esq., Wilmington, Del.

JAMES HAYWARD BACON, M. Am. Soc. C. E.*

DIED JUNE 4, 1924.

James Hayward Bacon was born in Chicago, Ill., on December 30, 1859. After his graduation from Harvard University in 1882, he began his professional career with the United States Corps of Engineers, and with the exception of three years, October, 1885 to October, 1886, as Draftsman and, later, as Division Engineer, with the Dubuque and Northwestern Railroad Company, and from May, 1892, to May, 1894, as Engineer in charge of the improvement of the St. Johns River by the County of Duval, Florida, he was connected with the Corps of Engineers until September 21, 1903.

From June, 1882, to July, 1884, Mr. Bacon served as Rodman on the improvement of Cape Fear River, in North Carolina, and from August, 1884, to April, 1885, he was engaged as Assistant Engineer and Assistant Superintendent under Maj. O. H. Ernst (now Maj.-Gen., U. S. A. (*Retired*)), M. Am. Soc. C. E., on the improvement of the Mississippi River between the Missouri and Ohio Rivers. From 1886 to 1892, he held the position of U. S. Assistant Engineer under Capt. W. H. Bixby (now Brig. Gen., U. S. A. (*Retired*)), M. Am. Soc. C. E., in charge of the survey of Lockwood's Folly River and, later, in the Florida District, under Capt. W. M. Black (now Maj.-Gen., U. S. A. (*Retired*)), M. Am. Soc. C. E., and Maj. J. C. Mallery, he had charge of surveys of the Manatee River, Charlotte Harbor, Tampa Bay, Sarasota Bay, St. Johns River, St. Johns Bar, and Ocklawaha River.

From March to December, 1895, as U. S. Superintendent of Dredging, he was engaged in dredging Charlotte Harbor under the late Maj. T. H. Handbury, M. Am. Soc. C. E.

From February to April, 1896, Mr. Bacon was engaged on the construction of the concrete bicycle track at Panama Park, Fla., and from May, 1896, to 1903, he served as U. S. Assistant Engineer in the Florida District in charge of surveys of the St. Lucie River, St. Johns Bar, Lower and Upper St. Johns River, Cumberland Sound Bar, and Biscayne Bay, and in connection with the various duties incident to the improvement of rivers and harbors in the District.

During the Spanish-American War, Mr. Bacon held a Commission as First Lieutenant in the 3d Regiment, Engineers, U. S. V. From September 1898 to March 1899, he was employed in designing gun and mortar batteries at Key West, Fla., and on the construction of batteries at the entrance to Tampa Bay, Fla.

On resigning from the Government service in 1903, Mr. Bacon accepted a position with the Grand Trunk Railway System of Canada and was assigned to the duty of selecting a terminal harbor for this System. Prince Rupert, British Columbia, was chosen as the site of the harbor. Mr. Bacon was then

* Memoir prepared by E. R. Conant, M. Am. Soc. C. E.

engaged on the construction of terminals and hotels, the laying out of streets, etc. At the outbreak of the World War, he returned to the United States and entered into contracting and engineering work in Virginia. At the time of his death, on June 4, 1924, he was a member of the firm of Bacon and Moore, with an office in Norfolk.

In 1909, Mr. Bacon was married to Bessie Tyson, of Jacksonville, Fla., who survives him.

He was an engineer of high standing, thorough in every project that he undertook, and will be remembered no less for his unquestioned integrity and kindly personality than for his professional work resulting in benefit to the general public.

Mr. Bacon was elected a Member of the American Society of Civil Engineers on January 3, 1900.

WALTER FRANCIS BALLINGER, M. Am. Soc. C. E.*

DIED DECEMBER 23, 1924.

Walter Francis Ballinger was born in Petroleum Center, Venango County, Pa., on August 13, 1868, and received his education at the Woodstown, N. J., Grammar School; the Philadelphia, Pa., Business College; the International Correspondence School, Scranton, Pa.; and Drexel Institute, Philadelphia, Pa.

Mr. Ballinger entered the architectural offices of Geissinger and Hales, in Philadelphia, in 1889, and in 1895 became a member of the firm which was continued under the name of Hales and Ballinger. He became the senior member of the firm under the name of Ballinger and Perrot in 1901, which firm in 1920 became The Ballinger Company.

He became interested in the possibility of reinforced concrete building construction when there were only a few such buildings in existence and he did much to aid in the development of this form of construction. He was also co-inventor of a new type of saw-tooth roof construction known as "Super-Span", which while it possesses the advantages of the natural illumination of that type, eliminates a large percentage of the columns usually needed.

Mr. Ballinger took an active interest in professional societies in connection with architecture and engineering. He was a member of the National Fire Prevention Commission, the American Society of Mechanical Engineers, Franklin Institute, Engineers Club of Philadelphia, Engineers Club of New York, Pen and Pencil Club, and the Society of Industrial Engineers.

Having been interested in all organizations for civic improvement, he was a member of the Philadelphia Chamber of Commerce, the Philadelphia Board of Trade, the Germantown and Chestnut Hill Improvement Association, the Fairmount Park Art Association, and the City Club of Philadelphia.

Mr. Ballinger was registered as an Architect and Professional Engineer in the States of Pennsylvania, New York, and New Jersey. His professional practice included the design and supervision of erection of commercial and industrial plants, institutional buildings, including hotel, hospital, and club buildings, as well as churches and schools.

As a member of the Methodist Episcopal Church, Mr. Ballinger was active in its work, as well as in that of local charitable organizations, having been a member of the Board of Temperance, Prohibition and Public Morals and of the Philadelphia Missionary Society of this Church. He was also a member of the Philadelphia Law Enforcement League and of the Board of Managers of the Pennsylvania Seamen's Friend Society and the Seamen's Church Institute.

* Memoir prepared by William R. Fogg, Secy., The Ballinger Co., Philadelphia, Pa.

He was a Mason, a member of the Blue Lodge, Chapter, Consistory, and Shrine, and, in addition, was a member of the Manufacturers Club, Penn Athletic Club of Philadelphia, North Hills Country Club, Lake Placid Club, and the Young Men's Christian Association.

Mr. Ballinger died in Philadelphia, on December 21, 1924, as the result of an automobile accident.

In 1896, he was married to Bessie M. Connell, who, with one daughter, Grace A., now engaged in social service work, and one son, Robert I., a member of The Ballinger Company, survives him.

Mr. Ballinger was elected a Member of the American Society of Civil Engineers on February 7, 1906.

JOHN BOGART, M. Am. Soc. C. E.*

DIED APRIL 25, 1920.

The death of John Bogart marks in the recollections of civil engineers, both young and old, the passing of one of the great engineering minds of the time. His life, from early youth to the age of 84, was devoted to the profession of engineering, and he leaves behind him a record of achievement in the inception, design, and construction of work, that has been excelled by few engineers of his generation.

John Bogart was of Dutch descent, his ancestors having come from Holland to the United States about 1641. They originally settled in Albany, N. Y., where he was born on February 8, 1836, the son of John Henry and Eliza Hermans Bogart.

He received his early education in the Albany Academy, which institution was noted for its thorough training. At that period, the two great prizes of the academic year were the Van Rensselaer Classical Medal and the Caldwell Mathematical Medal, given to the best student in each of those branches of study. Mr. Bogart was the first person awarded both these medals in one year.

From the Academy he went to Rutgers College, at New Brunswick, N. J., where many sons of Albany Dutchmen had received their collegiate education. He was graduated in 1853 with the degree of Bachelor of Arts. The College subsequently conferred on him the degree of Master of Arts and, in 1912, the degree of Doctor of Science.

Mr. Bogart began the study of law with the intention of practicing that profession, but on leaving college his health was delicate, and to secure the advantages of active exercise he entered at once the Corps of Engineers of the New York Central Railroad and was actively engaged for several years on the improvements of its lines. A large part of his duties was in connection with the construction of the direct road between Syracuse and Rochester, N. Y., through Clyde, Lyons, and Palmyra, which affected a saving of twenty miles as compared with the older line by way of Auburn, Geneva, and Canandaigua. This experience in engineering work established his choice of a profession.

His next service was as an Assistant in the Engineer Department of the State of New York, in which position he was employed on the works of reconstruction and enlargement of the canals in the Eastern Division of the State. For some time, he occupied one of the offices in the State House over which, thirty years afterward, he presided as State Engineer.

The construction of Central Park in New York, N. Y., was at that time being planned. This work involved important engineering in the roads, tunnels, bridges, drainage, and water system, as well as the artistic element of landscape treatment which appealed especially to Mr. Bogart's cultivated taste. Here, he was engaged until the beginning of the Civil War and became deeply interested in the development of urban and suburban park improvements. He was afterward connected with many such improvements in various parts of the

* Memoir prepared by Herbert Spencer, Esq., and Charles A. Pohl, M. Am. Soc. C. E.

United States, notably as Chief Engineer of the Brooklyn Park Commission in the construction of Prospect Park.

The urgent demand of the Government at the outbreak of the great Civil War for the services of the young men brought immediate response from Mr. Bogart and from his only brother, James Henry Bogart, who served through the war to the siege of Port Hudson, where, as Major of one of the New York regiments, he was killed while leading his troops. John Bogart was invited to act as an Engineer and served throughout the war, being stationed mostly in Virginia. He had charge, under Brig.-Gen. William H. Brewerton, of the construction of the heavy fortifications on the Rip Raps, an island in Hampton Roads, which, in connection with Fort Monroe, guarded the mile-wide channel from the ocean to the James River and to Norfolk, Va. He was present at the memorable engagement between the ironclad, *Merrimac,* and the *Monitor,* then called Ericsson's Battery, witnessing from the mast of one of the ships the fight which revolutionized naval warfare. He was also engaged in military operations at a number of other points during the war and at its close remained for a time in Government service. He was soon called, however, to the active duties of his profession in civil life and resigned in 1866.

While a detailed account of the work executed or investigated by Mr. Bogart would be too lengthy, the rather marvelous range of projects submitted to him for advice may be mentioned in order to show his grasp and understanding of engineering in its larger aspects.

In connection with the development of park areas, he served as Chief Engineer or as Consulting Engineer on work of great importance in many cities. As Chief Engineer of the Department of Public Parks in New York City, from 1872 to 1877, he constructed what are probably the best known parks in the United States. His work also included parks and boulevards in Albany, N. Y., New Orleans, La., Nashville, Tenn., Baltimore, Md., Chicago, Ill., Essex County, New Jersey, and Riverside Drive, New York.

Mr. Bogart served as Deputy State Engineer and Surveyor of New York State during 1886 and until the summer of 1887, when he resigned that position. He was elected State Engineer and Surveyor in the fall of that year, assuming office on January 1, 1888.

One of his early designs was for a bridge at Hartford, Conn., considered at the time to be one of the first flat arches in America. In 1887, Mr. Bogart was engaged as Chief Engineer in charge of the construction of the beautiful Washington Bridge over the Harlem River in New York. He also served as Advisory Engineer for the original Rapid Transit Commission of New York, and, as such, was engaged in the preparation of plans and contracts for the first subway system. He also prepared plans for tunnels under the Hudson River to bring those railroads that had terminals in Jersey City and Hoboken, N. J., into a proposed Union Station at 42d Street and Eighth Avenue, New York. The Steinway Tunnel to Queens, now operated as a part of the New York Subway System, was included in the plan for bringing the Long Island Railroad into the proposed Union Station. A bridge over Hell Gate and a

Connecting Railroad to Bay Ridge, Long Island, with a ferry to Bayonne, N. J. (substantially as built later), was projected as part of this design.

The foundations of some of the earlier of the large buildings in New York also engaged his attention, such as difficult problems of deep foundations, notably those of the Commercial Cable Building, Queens Insurance Building, Hanover Bank, Savoy Hotel, New York Life Insurance Building, New York Athletic Club, and St. Agnes Chapel.

In 1890, Mr. Bogart became much interested in hydro-electric development which was then in its early stage, and was in touch with most of the projects reported on or constructed. He was appointed Consulting Engineer of the Cataract Construction Company, a subsidiary of the Niagara Falls Power Company. At that time, little was known of power transmission, but certain European developments had attracted some attention; with the view of utilizing some method for the transmission of the vast power of Niagara, he investigated such European installations as could then be found. Accompanied by Mrs. Bogart and the late Francis Lynde Stetson, Vice-President of the Niagara Falls Power Company, he traveled over a large part of Europe studying existing methods of power generation and transmission. Four methods of water-power transmission were found to be in use: (1) By means of manila or wire rope, in some instances taking power nearly a mile, which method was that in most general use; (2) by hydraulic transmission through pipes; at Geneva, Switzerland, electric lighting dynamos were operated by water piped under pressure through city streets; (3) by pneumatic transmission, which method was in extensive use in Paris, France, Birmingham, England, and elsewhere; and (4) by electricity.

In all France, they found only three examples, and, in Switzerland, only one example, of electric transmission in actual use. At Domène, opposite the Grand Chartreuse in the Dauphine Alps, the power for a paper-mill was drawn from a glacial stream in the mountains four miles away. Here was found the precedent which largely influenced the final decision as to the system to be adopted at Niagara. Mr. Bogart was actively engaged in the planning and construction of this pioneer development in America from its inception to its completion.

The tremendous field of hydro-electric development and the possibilities of electric transmission appealed very strongly to him. In 1899, he investigated the possibilities of utilizing the power of the St. Lawrence River at Massena, N. Y., and jointly with Messrs. Kincaid, Waller, and Manville, of London, England, he designed and built the plant of the St. Lawrence Power Company at that place.

As a Consulting Engineer on hydraulic developments, Mr. Bogart's reputation was now established, and brought him extensive work in various parts of the country. He served as Engineer of the plants of the Cascade Power Company in British Columbia, the Lake Superior Power Company, the Knoxville Power Company, the Youghiogheny Power Company, the Niagara, Lockport, and Ontario Power Company, and the Sault Ste. Marie Power Company.

He also designed and built the plant of the Atlanta Power Company at Bull Sluice, Ga., and many others.

Probably Mr. Bogart's greatest contribution to hydro-electric development was as Chief Engineer of the Chattanooga and Tennessee River Power Company, on a 60 000-h. p. plant in the Tennessee River near Chattanooga, which was the first installation using private capital to improve the navigation of a river under Government control in return for the power created. This work was particularly difficult on account of the great variation in head on the turbines and the difficult foundations for the dam. It was found necessary to resort to reinforced concrete pneumatic caissons in making the excavation and afterward to incorporate these as part of the dam. As far as known. this was the first instance where pneumatic caissons had been used for this purpose.

In 1913, Mr. Bogart formed a partnership with Charles A. Pohl, M. Am. Soc. C. E., which was continued until his death on April 25, 1920. The business is still maintained by Mr. Pohl under the firm name of Bogart and Pohl.

In 1870, Mr. Bogart was married to Emma Cherrington Jefferis, of West Chester, Pa., who survives him.

He served on many boards and commissions, of which the following is a partial list: Nicaragua Canal Commission, Railroad Terminal Commissions of Buffalo, N. Y., and Toronto, Ont., Canada, Croton Aqueduct Commission, New York State Board of Health, etc. He was appointed a member of the Municipal Art Commission of New York City, and was the only civilian member of the Board of Engineers appointed by the President of the United States to examine the feasibility of constructing a deep waterway from Chicago, Ill., to the Gulf of Mexico. Mr. Bogart was also delegated by the President to represent this country at the International Navigation Congresses held at Düsseldorf, Germany, in 1902; at Milan, Italy, in 1905; and at St. Petersburg, Russia, in 1908. At Philadelphia, Pa., in 1912, he served as Chairman of the Inland Section of the Congress.

He was a Lieutenant-Colonel of Engineers of the National Guard of New York State and served for ten years on the Staff of Gen. Charles P. Roe.

Mr. Bogart was employed as expert in many Court cases. His work was of such varied nature that his testimony was equally varied, and his clearness of expression, uniform courtesy, and good humor made it extremely valuable to his clients.

Mr. Bogart believed the truly great engineer should be able to harmonize and correlate all phases of engineering so as to make his work of the greatest service to mankind. The following extract from a speech delivered by him before the Engineers' Club of Rochester, N. Y., serves to illustrate his ideas on the question of specialized engineering:

"As perhaps the oldest member of the profession here to-night (though indeed you are making me feel young again by your courtesy), I may recall the difference in the larger appreciation of the work of the Engineer. It was always necessary, essential, fundamental in the accomplishment of great undertakings, but not so very long ago the men who were talked about, recognized by the great public in connection with material progress and the success of great undertakings, were the men who managed the finances. In fact,

Engineers used to be willing to permit their functions to appear to be subordinate. The Engineer had not 'found himself'. He did not quite appreciate the fact, or, at least, did not insist upon large recognition of the fact that what he gave to an enterprise, was the art, the skill, the 'know-how', without which neither money nor material, nor energy could be so put together, each in its right relation, like the various parts of a timepiece, and thus make the enterprise a working success.

"I think we may fairly say that conditions have been changed. The industrial history of the recent years of progress has been in large part made by the achievements of Engineers. You do not need a recital of instances. They stand out large, and the world is not unconscious of these facts. The world desires to know about feats of Engineering, and, gentlemen, we should not, in justice to each other, hide our brilliant light.

"The phases of Engineering have become wonderfully varied, not only in the great divisions, mechanical, mining, electrical, all sub-phases of civil engineering, but all these demanding special study of interdependent arts. I sometimes fear that some of us specialize too much for the general good. We must not forget that each specialization is only a part, and that somebody must combine all essentials into such harmonious results as should characterize good engineering, and we must insist that the one who accomplishes final results is the great engineer."

During his long and interesting career, Mr. Bogart was associated with most of the great engineers of the Nineteenth Century, and his genial disposition, combined with his remarkable fund of knowledge, drew men to him for advice on the most diversified engineering subjects. This is the more remarkable in this day of highly specialized engineers. In his career, however, Mr. Bogart seems to have covered the field of engineering work, from railroads, canals, water-works, tunnels, parks, and bridges, to great hydro-electric developments, dams, and inland waterway and irrigation projects. He was in the true sense an organizer and, as such, in great demand. His profession was a labor of love and he gave to his work the painstaking care which was marked more by the desire to serve his fellow-beings than for financial return.

The Society owes a great deal to the memory of Mr. Bogart for his untiring work in its early days. He served as a Director from 1873 to 1875, as Treasurer from 1876 to 1877 and from 1891 to 1894, and as Secretary from 1878 to 1890, and was largely responsible for shaping many of the ideas and policies which have served to give the Society its present standing.

Mr. Bogart was elected a Member of the American Society of Civil Engineers on February 17, 1869.

DANIEL BONTECOU, M. Am. Soc. C. E.*

DIED JULY 14, 1924.

Daniel Bontecou, the son of William Ely and Caroline C. (Thayer) Bontecou, was born on September 14, 1851, at Springfield, Mass., the home of his grandfather, Daniel Bontecou. His early boyhood was spent in Weymouth, Mass., and he attended the English High School of Boston. Later, the family moved to New York, N. Y., and at the age of 16 years Mr. Bontecou, through his own efforts, entered the College of the City of New York, from which he was graduated in 1871, second in his class, with the degree of Bachelor of Science.

He began his professional career in New York, in the Department of Public Works, where he was employed from 1871 to 1873. He was then appointed Assistant Engineer with the New York Central Railroad Company and assigned to duty at Fonda, N. Y., where he was in charge of 28 miles of the four-tracking of that System, and also of work at the 59th Street Terminal in New York. On the completion of this work, in 1874, he became Assistant Engineer on the location survey for the railway line between Albany, N. Y., and the Hoosac Tunnel.

In 1876, Mr. Bontecou was appointed Assistant to the late Elnathan Sweet, M. Am. Soc. C. E., then State Engineer of New York, on the New York State Canals, in which position he continued until 1878, when he returned to the employ of the City of New York, and was in charge of the construction of the Madison Avenue Bridge and of plans for the Washington Bridge and other bridges across the Harlem River. His service with the City of New York continued until 1881, at which time he removed to Kansas City, Mo., and formed an engineering partnership with the late William B. Knight, M. Am. Soc. C. E., under the firm name of Knight and Bontecou, to carry on a general civil engineering practice.

In 1883, Mr. Bontecou was appointed Chief Engineer of the Kansas City Belt Railway Company, in which position he continued until 1886, when he became Chief Engineer of the Grand Avenue Railway Company, of Kansas City. During 1889, he was called to Washington, D. C., and appointed Chief Engineer for the Capital Traction Company of Washington, on the construction of the cable-operated street car lines and power plant of that city, where he remained until their completion and operation in 1890. Mr. Bontecou then became Chief Engineer of the Kansas City, Fort Scott, and Memphis Railroad and Associated Companies, including the Kansas City Belt Railway, which position he retained until 1901. From 1901 to 1918, he was engaged in private practice in Kansas City as Consulting Engineer, and in this capacity he was retained in connection with numerous projects of many different types and clients, including the following: Viaduct plans for the Metropolitan Street Railway Company; the Kansas City Lighting Company; the Central Avenue

* Memoir prepared by Robert Ridgway, President, Am. Soc. C. E., and Eugene G. Haines, M. Am. Soc. C. E.

Power House; the Kansas City Stock Yards; the design and construction of Kansas River bridges, and water supply; the Capital Traction Company of Washington, Pennsylvania Avenue Cable Lines; the Kansas City Cement Company and the Hawkeye Cement Company; the Kansas City and Western Railway Company on valuation and line improvements; steel and concrete buildings, power installations, etc., for the United Zinc and Chemical Company; reports to the Fidelity Trust Company on the Kansas City and Missouri Interurban Railway, including plans and expenditures; the Oklahoma, Kansas and Missouri and Okmulgee Northern Railways; the Kansas City and Missouri River Navigation Company, traffic studies and warehouses; the St. Louis and Santa Fé Railroad on revision of valuation of the Arkansas Lines; report to creditor's banks on the cost of building the Galveston Causeway; and reports to railway companies and others on the cost and details of reconstructing fifteen bridges after the Kaw River floods in 1903.

Mr. Bontecou was a member of the Advisory Board of Engineers on the matter of industrial preparedness, representing the State of Missouri and working under the Naval Consulting Board in 1916. He also served the Government in other ways, reporting, in 1918, to the U. S. Shipping Board on "The Availability of Inland Waterways to Relieve Railroad Congestion", and to the Inland Waterways Commission on "The Traffic of the Mississippi River above St. Louis"; he also served as District Engineer with the Housing Corporation in charge of the proposed developments at Perth Amboy, N. J.; Port Penn, Del.; and Sheffield, Florence, and Tuscaloosa, Ala. He was a member of the Engineers Club of New York.

It is not easy to define the personal characteristics which, taken in the aggregate, mark a man as different from many others, for they represent only the outward reflection of the rays from many qualities, often elusive and not pronounced, but where many good qualities are combined in one man, he is known as a man of "fine character", and such was Daniel Bontecou. A composite picture, drawn by quotations from letters received since his death, reveals him as: "A very quiet man" of "pleasant manner", and "always a perfect gentleman"; "one you could go to for advice at any time and always meet with a kindly word and good advice". He was "an able engineer", "studious", "one of the most thorough men I have ever met", "a clear thinker", and, "one who loved to work out intricate details". He was a "close observer" and "quickly came to know the worth of the men under him and their ability". He "inspired confidence", and those with whom he became associated "learned to know and love the man" as one "inspired by the highest motives" and of "the very highest ideals and character", and they placed "great faith in him as an engineer". He was "a most devoted son", "a loyal citizen", and "a warm personal friend". What a tribute this is to the memory of a man; and yet it has all been said, in one brief and sweetly beautiful sentence, by one who knew him best and who kept pace beside him for thirty-nine years as his wife: "He died, as he had lived, bravely and patiently; and he leaves a wonderful memory for his children, and for me."

Mr. Bontecou was married in 1885, at Falmouth, Mass., to Nathalie Holdrege, of Irvington, N. Y. His death occurred at Mamaroneck, N. Y., where he had resided since his return to the East, and he is survived by his wife and five children, Daniel Jr., Russell, Frederick H., Helen, Nathalie Bontecou Crocker, and by six grandchildren.

Mr. Bontecou was elected a Member of the American Society of Civil Engineers on November 5, 1879. He served as Director from 1896 to 1898, and as Vice-President in 1915 and 1916.

JOHN GRIFFITHS BROWN, M. Am. Soc. C. E.*

DIED OCTOBER 22, 1917.

John Griffiths Brown, the son of James and Mary A. Kerr Brown, was born on June 1, 1873, at Philadelphia, Pa.

He was educated in the public schools, and was apprenticed in architectural wood-carving in his father's shop. His contact in his work with architects and engineers developed in him the ambition to become an engineer, and on finishing his apprenticeship he secured employment with the United States Engineer Corps on survey work along the Delaware River, and, subsequently, a position in a surveying corps on the New York subways.

In 1901, Mr. Brown became a Cement Inspector on the Atlantic Avenue Improvement in Brooklyn, N. Y., which position he held until, in 1903, he was appointed Inspector of Cement and Concrete Material for the Philadelphia Rapid Transit Company, when it entered on the construction of its Market Street Subway and Elevated Line.

In this position he pursued his study of structural design as well as of the technique of concrete making, and, in 1904, became General Manager of the Unit Concrete-Steel Frame Company of Philadelphia, engaged in the construction of buildings of pre-cast reinforced units.

In 1906, Mr. Brown established himself in business as a Designer and Contracting Builder, specializing in concrete industrial buildings—mills and factories—his service covering the whole range of the study of plant requirements, the production engineer's field, surveys of sites, designs of buildings, and construction and equipment.

Among the many operations prosecuted by him are the Fayette R. Plumb Plant of sixteen buildings, St. Louis, Mo.; the Savage Arms Plant, Utica, N. Y.; the Grellet Collins Building, Hale and Kilburn Factory, Electric Storage Battery Company Building, and William F. Read Sons Building, Philadelphia; the American Malleable Company's Plant, Buffalo, N. Y.; the Ingersoll Watch Company, Trenton, N. J.; the Mack-Saur Motor Company, Plainfield, N. J.; the Wall Rope Works, Beverly, N. J.; and the Jeansville Iron Works, Hazleton, Pa.

Mr. Brown's business policy was a far-sighted one—to obtain clients through notable devotion to their interests. His contracts for construction as well as for design were on a percentage basis, but he studied and strove to economize in his client's interest. He built up an efficient organization of specialists, and systematically provided technical instruction for his employees by professional teachers and experienced practical men, furnishing the books and instruments as well as the teachers, the instruction covering practical

* Memoir prepared by Henry H. Quimby, M. Am. Soc. C. E.

points of work, design of structures, and construction equipment, inspection, architecture, and psychology. He called for intelligent as well as faithful devotion to work, believing in the necessity of thorough vocational training.

Mr. Brown held and presided at periodical organization meetings, attended by the heads of departments, foremen, engineers, draftsmen, office employees, and apprentices, at which all branches of the business were freely discussed, differences were ironed out, and co-operation was promoted.

He was of a genial, kindly disposition, with a host of friends, energetic in technical society activities as well as in his professional work; he placed high value on character in his associates and on uprightness in his employees, investigating thoroughly the habits of an applicant before employing him.

Mr. Brown was a member of the American Society for Testing Materials, and of the Engineers' Club and the Manufacturers' Club of Philadelphia.

He was married to Margaret B. Taylor, on November 2, 1901, who survived him for several years. They had no children.

Mr. Brown was elected an Associate of the American Society of Civil Engineers on April 3, 1906, and a Member on September 3, 1913.

WILLIAM DEXTER BULLOCK, M. Am. Soc. C. E.*

DIED APRIL 30, 1923.

William Dexter Bullock was born in Rehoboth, Mass., on April 17, 1850, the son of William R. and Hannah G. (Carpenter) Bullock, both of whom were of New England descent. Mr. Bullock was a graduate of Union College, Schenectady, N. Y., and received the degree of Civil Engineer in 1871.

Taking up office and field work with the Delaware, Lackawanna, and Western Railroad Company for a season, he was afterward employed in the office of the late James B. Francis, Past-President, Am. Soc. C. E., of Lowell, Mass., and in the City Engineer's Office in Lowell. In 1872, he assisted in the surveys in Washington Territory for the Northern Pacific Railroad, and, in February, 1873, he entered the City Engineer's Office in Providence, R. I., with which he was destined to be connected for the remainder of his life. For some years, he was engaged in a general way with the various branches of municipal work, but having long cherished an ambition to be a bridge engineer, he was much pleased when, in 1878, he was given charge, as Engineer of Bridges and Harbor, of the bridge and harbor work of the city by the City Engineer, the late Samuel M. Gray, M. Am. Soc. C. E. This position Mr. Bullock retained until his death.

Owing to the location of the City of Providence, the Bridge and Harbor Department is one of the important Divisions of the City Engineer's Office. The Woonasquatucket and the Moshassuck Rivers flow through the city, meeting at the center to form the Providence River. This river, together with the Seekonk River which bounds the city on the east, forms the head-waters of Narragansett Bay. The Bridge and Harbor Department not only performs the engineering required in connection with the construction and maintenance of the bridges over these streams, navigation in the Providence River, and the various City docks, but also maintains the operating forces for the draw-bridges and a construction force which does all the maintenance work, except the major contract work of reconstruction. In the forty-five years during which Mr. Bullock had charge of the Department many of the old bridges were entirely renewed or reconstructed and new ones were built, so that, at the time of his death, the Department was responsible for three swing draw-bridges and about forty fixed bridges which embraced examples of almost every type of modern bridge, including three substantial granite arches designed to harmonize with the surroundings at Roger Williams Park.

These structures show many examples of Mr. Bullock's ingenuity, especially two of the draw-bridges which are equipped with automatic safety gates of his design. By means of these gates, travel on the bridge may be controlled from the operating room on the draw-span.

About 1890, the plans for the present railroad facilities in the center of the city were adopted. The Bridge and Harbor Department supervised a part of

* Memoir prepared by Herbert E. Sherman and Elmer W. Ross, Members, Am. Soc. C. E., and Robert L. Bowen, Assoc. M. Am. Soc. C. E.

this work which involved the construction of long granite retaining walls for the Woonasquatucket River, the removal of the mud and silt from the "Cove Basin"—a circular tide-flowed basin, 30 acres in extent—and the filling in of the basin with gravel to the elevation of the surrounding streets.

All the City docks were constructed under Mr. Bullock's direction. One built for the Highway, Sewer, and Water Departments was of wooden pile construction. The "Municipal Wharf" so-called, which was started in 1911 and completed in 1915, involved the construction on the harbor line of a granite quay wall, 3 000 ft. long and 41 ft. high. Back of the wall a large area was filled and graded, and railroad tracks, sidings, etc., were installed.

Between 1878 and 1923, the purchasing power of the dollar varied so widely that a statement of the cost of the work done under Mr. Bullock's supervision during that period does not adequately convey an idea of its amount. It may be of interest, however, to note that during that time about $2 200 000 was spent on bridges, $500 000 on dredging, and $1 000 000 on City docks.

In addition to his work for the City of Providence, Mr. Bullock also served as Chief Engineer for the State Harbor Improvement Commission of Rhode Island during its existence from 1911 to 1918. This Commission was appointed by the General Assembly to provide additional facilities for commerce by water in Providence and Pawtucket, R. I. In Providence, it constructed a large, thoroughly modern pier, with a steel pier shed, equipped with railroad connections, immigration facilities, and cargo-handling apparatus. In Pawtucket, the Commission constructed a granite quay wall, 700 ft. long, graded the area in back, and built a large steel freight shed. This work involved the expenditure of about $1 000 000.

On October 17, 1879, Mr. Bullock was married to Annie A. Taft, of Pawtucket, who died in Providence on October 31, 1899. On February 26, 1902, he was married to Florence S. Clapp, of Providence, who, with a son and a daughter, survives him.

He was a man of high principles and positive convictions, of marked ability and judgment, an agreeable associate, and a loyal friend. In 1886, he represented the City of Pawtucket in the Rhode Island House of Representatives. He attended the Central Congregational Church in Providence.

Mr. Bullock was elected a Junior of the American Society of Civil Engineers on September 5, 1887, and a Member on July 4, 1888.

CLINTON SUMNER BURNS, M. Am. Soc. C. E.*

DIED APRIL 1, 1924.

Clinton Sumner Burns was born at Waverly, Iowa, on October 26, 1871. He was the son of Herman H. and Laura Root Burns.

His early education was obtained in the public schools at Port Byron, N. Y., where he lived with his grandmother, his mother having died when he was nine years of age. He was graduated from the Port Byron Academy at the age of twenty and succeeded in winning a scholarship at Cornell University which he entered as an engineering student. While attending Cornell, he acted during vacations as Assistant City Engineer of Niagara Falls, N. Y.

After two years at Cornell, Mr. Burns went to California to enter Leland Stanford University. Before leaving Cornell, he had taken a Civil Service examination for the position of State Leveler for New York State and after arriving in California was notified of his appointment, his being the highest grade among a number of applicants. He returned to New York to accept this position and was engaged on work on the Erie Canal and on other State canals. At the end of the year, he returned to Stanford University from which he was graduated in 1897 with the degree of Bachelor or Arts in Engineering.

Mr. Burns secured his education and technical training at considerable personal effort, having sought employment at the various schools he attended as a means of defraying his expenses. While at Stanford University, he held the position of Assistant City Engineer of Palo Alto, and, on Saturdays and at odd times, established grades and made lot surveys. During his vacations, also, he secured engineering engagements, thus paying his way through the University.

After his graduation, Mr. Burns went to Kansas City, Mo., where he entered the employ of the City Park Department, under Mr. George Kessler. He held this position until April 1, 1898, when he and Robert E. McDonnell, M. Am. Soc. C. E., a classmate at Stanford University, formed a partnership under the firm name of Burns and McDonnell and began the private practice of municipal engineering, specializing in water supply and sewerage.

In 1919, the name of the firm was changed to Burns and McDonnell Engineering Company, four of the older employees having been admitted to membership in the organization. In 1923, a branch office was inaugurated at Los Angeles, Calif., to carry on the same class of work. Mr. Burns died on April 1, 1924, on the twenty-sixth anniversary of the establishment of the firm.

He gave his personal attention to the appraisal of a large number of water-works plants that changed from private to municipal ownership, the more important being those at Council Bluffs and Des Moines, Iowa, Everett, Wash., Billings, Mont., and Riverside, Calif. He lived at Riverside for about a year, personally planning and directing the rehabilitation of the water-works plant.

Mr. Burns was of a mechanical turn of mind, given to research and investigations of various kinds, and he often maintained testing apparatus and

* Memoir prepared by Robert E. McDonnell, M. Am. Soc. C. E.

equipment at his home for the special study and analysis of engineering problems. Several patents were granted to him for various devices, one a joint for sewer pipe, another for some special railroad equipment, etc. These patents were sought and secured not with a view of financial profit, but more for the love of and satisfaction gained in the work of investigations.

His clear thinking, love of mathematical problems, and analytical treatment of all engineering questions, was a matter of much comment on the part of his associates and all those who knew him personally. His profession called him to all parts of the country and involved work in practically every State in the Union.

Mr. Burns was a frequent contributor to the engineering press and had written a number of papers on valuation questions, of which that entitled, "The Valuation of Sewerage Systems", is one of the few—perhaps the only published article—covering this particular class of utilities. The paper relates to personal experiences during the appraisal of a number of sanitary sewerage systems throughout the Southern States, where such utilities are frequently owned by private companies. He was also the author of a pamphlet entitled, "The Relationship between Finance and Depreciation", and another pamphlet descriptive of the Sultan Water Supply of Everett, Wash., a gravity water supply project diverting the Sultan River through several tunnels and under river crossings in a 28-mile pipe into the City of Everett.

In 1905, nearly twenty years ago, Mr. Burns wrote an interesting article entitled, "The Diary of a Young Engineer", in which he recorded anecdotes and the early experiences of a young engineer struggling past the first few mile-posts of his professional career. These experiences show that at an early age he had a high conception of the ethics of his profession and his article closes with the comment: "The practicing engineer cannot hope to secure much business along the branch of the consulting profession until he has acquired a reputation of absolute honesty, untainted by commissions or connections with construction companies." He further states in this article that the young engineer may give his terms and if he is true to his profession his price will be high enough to enable him to do the work thoroughly. This interest in professional fees continued throughout his life, for he was Chairman of the National Committee of the American Association of Engineers on Services and Fees for Practicing Engineers.

At the time of his death, Mr. Burns was serving Kansas City as one of the five members of the Board of Review on the plans for its $11 000 000 water plant, which important improvement he had been actively urging for many years.

He became at an early age a member of various engineering organizations, having been one of the founders of what is now The Engineers' Club of Kansas City. He was elected a member of the New England Water Works Association in 1909 and, for several years, had been a member of the American Institute of Consulting Engineers. He was also an Honorary Member of the Sigma Xi, having been elected in 1905 from the Stanford University Chapter.

Locally, Mr. Burns was a member of several civic organizations. His chief recreation was golf, and he was a member of the Milburn Golf and Country Club and the Community Golf Club. His daughter, Miriam, was City Champion of Kansas City in 1924, State Champion of Missouri, and, in 1923, won the championship in the Women's Western Golf Tournament, and Mr. Burns took keen delight in following her contests. Besides his daughter, his wife, Mrs. Mabel M. Burns, survives him.

Mr. Burns was elected an Associate Member of the American Society of Civil Engineers on February 1, 1899, and a Member on January 3, 1905.

ALEXANDER MOSBY CLAYTON BYERS, M. Am. Soc. C. E.*

DIED APRIL 9, 1924.

Alexander Mosby Clayton Byers was born on the family plantation called "Glenwood", near Batesville, Ark., on December 11, 1857, the son of Judge William Byers, a distinguished lawyer, and Emily Burton Byers. Genealogies of both sides of his family reveal the names of hero ancestors who served in all the American wars, including the Revolution.

Mr. Byers' education was acquired at private schools in Batesville, at Arkansas College, and at the University of the South, at Sewanee, Tenn. Among his instructors at the latter institution were the late Gen. William C. Gorgas, Gen. E. Kirby Smith, and Bishop Quintard.

Soon after the completion of his education Mr. Byers started work on that particular phase of engineering to which he gave most of his life—railway location and maintenance. His first work was with the old Kansas City, Springfield and Memphis Railroad Company. Immediately thereafter, during parts of 1879 and 1880, he was engaged in land surveying in Texas.

From 1881 to 1887, he was employed by the Mexican Central Railroad Company, successively, on location, construction, and maintenance. During the earlier part of this period, on three separate occasions, while he was on the preliminary surveys, Mr. Byers covered afoot the long distance from the City of Mexico to the United States Boundary.

For the next two years he was Assistant Engineer on the Panama Railroad, during the French régime, under the late Ferdinand de Lesseps. The failure of the canal project in 1889 seriously affected the railway, and Mr. Byers returned to Mexico, where he was immediately entrusted with the location and construction, for the Mexican Southern Railroad Company, of a line over exceedingly difficult terrain—the descent from the table-lands to the tropics. On the completion of this project, he rejoined the staff of the Mexican Central Railroad Company, and served for ten years as Engineer of the Maintenance Department of that line.

From 1902 to 1905, Mr. Byers was engaged in engineering and contracting work, mainly for the Dos Estrellas Mining Railroad Company in the State of Mexico, Mexico, and for the Vera Cruz and Pacific Railroad Company in the State of Vera Cruz, Mexico. Following these engagements, he was Chief Engineer of La Dicha and Pacific Railroad Company. Later, he became Superintendent of Tracks, Bridges, Buildings, and Water Service for the Tehuantepec National Railroad of Mexico.

Due to the troubled state of affairs in Mexico, Mr. Byers returned to the United States in 1914, and practiced his profession in Nevada. In 1915, he again answered the call of the tropics and went to Guatemala, as Engineer on construction work and, later, during 1916 and 1917, to Cuba, returning to the United States on its entry into the World War. During the war, Mr. Byers made valuable confidential reports to the War Department.

* Memoir prepared by Walter F. Winton, Affiliate, Am. Soc. C. E.

For two years preceding his death, ill health compelled him to forego the active life to which he had been accustomed, and he therefore devoted his time to consulting work in San Francisco, Calif.

He was married on May 11, 1891, to Emma O. Stinson, who survives him.

The correspondence file left by Mr. Byers reveals the keen regret of his various employers when he left their service—a most striking testimonial of his technical ability.

He loved adventure and had traveled widely. He was a great hunter, and his reminiscences included tales of buffalo shooting on the plains of Texas. An intimate friend pays the following tribute: "Byers was a most companionable man. He loved a joke, saw the humor of all situations, was untiring physically and of a resilient spirit. His generosity was boundless. He tied his friends to him with strong bonds".

Throughout his life, Mr. Byers was a member of the Protestant Episcopal Church.

In the death of this kindly, talented, Christian gentleman, the Engineering Profession has sustained a great loss.

Mr. Byers was elected a Member of the American Society of Civil Engineers on November 6, 1907.

HENRY MARISON BYLLESBY, M. Am. Soc. C. E.*

DIED MAY 1, 1924.

Henry Marison Byllesby, the son of the Rev. De Witt Clinton Byllesby and Sarah (Mathews) Byllesby, was born in Pittsburgh, Pa., on February 16, 1859. He was of English descent, his American lineage on his mother's side dating to 1620 and on his father's side to 1789.

Mr. Byllesby took a preparatory course in the Western University of Pennsylvania, from 1871 to 1873, and a course in Mechanical Engineering at Lehigh University from 1873 to 1877, leaving at the end of his Junior year to engage in practical work with the Weston Shops at Newark, N. J.

In November, 1879, he was employed as a Draftsman by Robert Wetherill and Company of Chester, Pa., leaving that firm on June 1, 1881, to enter the employ of the Edison Company, where he was engaged as a Draftsman on the plans of the First District Pearl Street Station of the original Edison Electric Illuminating Company of New York. From November, 1882, to February 1, 1885, he was on the general Engineering Staff of the Edison Company, engaged in installation work in the United States and Canada. Thereafter until November 1, 1885, Mr. Byllesby represented the firm of Robert Wetherill and Company as Eastern Manager. He was then engaged, with several others whose names have since become familiar in the electrical industry, by the late George Westinghouse, M. Am. Soc. C. E., who at that time was conducting his electrical experiments, as a personal venture, in the shops of the Union Switch and Signal Company, which he then controlled. Soon after the Westinghouse Electric Company was organized, with Mr. Byllesby as Vice-President and General Manager. The rapid development of the electrical industry and the prominent place assumed in it by the Westinghouse Company, which soon absorbed the Union Switch and Signal Company, gave him ample opportunity to exercise his talents, and resulted in his taking out, either personally or as joint inventor, numerous patents relating to electrical lighting apparatus, and also in his representing his Company in Europe from May 11, 1889, to September 1, 1891.

In December, 1891, he resigned and, on February 1, 1892, was engaged by the Thomson-Houston Electric Company, becoming, on April 1, President of the Northwest Thomson-Houston Company, a subsidiary organization, with headquarters at St. Paul, Minn. He remained there until the spring of 1895, the Company at that time being absorbed by the General Electric Company. For the next seven years Mr. Byllesby's energies were occupied in developing various important hydro-electric and other properties throughout the Northwest.

On January 1, 1902, he moved to Chicago, Ill., and organized the H. M. Byllesby and Company, Incorporated, of which he was President. Originally established as an engineering firm, the Company soon became interested in the financing, design, construction, operation, and management of electric and

* Memoir prepared by Bion J. Arnold, M. Am. Soc. C. E.

gas properties. During recent years, it has devoted itself to the underwriting and distribution of investment securities, the Byllesby Engineering and Management Corporation having taken over the operating, managing, and engineering work. Mr. Byllesby was either President or Director of many of the large utility companies of the United States, and was the pioneer in the establishment of the customer-ownership policy which is now so popular.

Notwithstanding his numerous affiliations, he found time to aid in civic work, and served with distinction as President of the Chicago Civic Federation. As Chairman of the Executive Committee of the Chicago Branch of the National Security League, he was largely responsible for inaugurating the rousing patriotic speaking campaign in Chicago, the West, and the Northwest, during the fall and winter of 1917.

Having offered his services at the entrance of this country into the World War, Mr. Byllesby was commissioned as Major in the Recruiting Division of the Signal Corps, U. S. Army, on November 15, 1917. Later, he went overseas, was promoted to the rank of Lieutenant Colonel, and served as the London representative of the American Expeditionary Forces, having charge of purchases in London and the Scandinavian countries. On December 19, 1918, he was honorably discharged and returned to his civilian duties. He was awarded the Distinguished Service Order by the British Government, and, on August 21, 1922, the Distinguished Service Medal of the United States, for "exceptionally meritorious and distinguished services." In 1923, he was commissioned a Colonel, Corps of Engineers, in the U. S. Officers' Reserve Corps.

In October, 1923, on account of serious heart trouble, Colonel Byllesby was compelled to drop all business cares, and he and Mrs. Byllesby spent the winter in California, returning to Chicago on April 17, 1924. He was in good spirits and confident that he would soon be able to resume his business duties. It was hoped that he was recovering when his death came unexpectedly on May 1, in the office of a dentist, where he had gone for treatment, accompanied by his nurse.

Funeral services were conducted in St. Paul's Protestant Episcopal Church, Chicago, on May 3, at which the Right Rev. Herman R. Page, Bishop of Michigan, an old and intimate friend, and a former Rector of St. Paul's, spoke on the life and personality of Colonel Byllesby. Following the services the funeral party was taken by special train to Lake Geneva, Wis., where the burial rites were conducted with full military honors; an honor guard from Fort Sheridan, the cadets of the Northwestern Military Academy, and members of Byllesby Post of the American Legion, participated.

Colonel Byllesby was married to Margaret Stearns Baldwin, of Roselle, N. J., on June 15, 1882, who survives him. They resided in Chicago and Holly Bush House, Lake Geneva, Wis.

He was a member of the American Society of Mechanical Engineers, the Western Society of Engineers, the Public Policy Committee of the National Electric Light Association, and a Fellow of the American Institute of Electrical Engineers. He was also a member of the leading clubs of Chicago,

New York, Minneapolis and St. Paul, Minn., Louisville, Ky., Portland, Ore., and San Francisco and San Diego, Calif.

Colonel Byllesby had great civic and National pride, and gave freely of his strength and means to all movements for education, social betterment, and the advancement of patriotism. In his death the country has lost an energetic and valuable citizen; the utility field, a liberal and progressive leader; the Engineering Profession, an able member; and those who knew him intimately, a generous and whole-souled friend.

Colonel Byllesby was elected a Member of the American Society of Civil Engineers on June 1, 1887.

ALBERT CARR, M. Am. Soc. C. E.*

DIED JUNE 18, 1924.

Albert Carr was born at Collinsville, Conn., on April 13, 1859, the son of Rodney and Esther (Pettibone) Carr.

He was graduated from Yale University in 1883, and after two years of general railroad practice, entered the rapid transit field in 1885 as Assistant Engineer of the Brooklyn Elevated and Union Elevated Railroads, in charge of foundations, track, iron erection, 36th Street Terminals and Yards, litigation surveys, and all field work including the construction of the Broadway Cable Road.

In 1893, he was given charge of the construction in New York of the Columbus Avenue Cable Road, Lexington Avenue and 116th Street Underground Electric, and, after 1895, of all field work connected with the Metropolitan Street Railway Company, including the construction of 75 miles of underground electric system. In 1900, he became Division Engineer of the Rapid Transit Commission, in charge of construction of the subway between the City Hall and 41st Street.

In 1904, Mr. Carr was chosen as Manager of Construction and Chief Engineer of the Mexican Light and Power Company, for which he built a power-house, tunnels, pipe lines, etc., including the foundations of the great earth dam at Necaxa, this installation supplying the City of Mexico, El Oro, and Pachuca, Mexico.

In 1906, the California disaster made necessary the prompt reconstruction of the United Railways of San Francisco. Here, Mr. Carr achieved great success in the unique problem of a rush order for a new and extensive street railway system for a great city. His next assignment was the construction of a dam, power-house and allied works for the Colorado Power Company, after which he, in company with the late George S. Rice, M. Am. Soc. C. E., made the necessary investigations and a report on the Lacroze Subway Project in Buenos Aires, Argentine Republic. Following this, his work was in New York City almost continually, for some years as Technical Counsel in litigation to the Interborough Rapid Transit, and during his last two years as Chief Engineer of Powers-Kennedy Corporation, Contractors for the Queensborough-Eighth Avenue Extension under 42d and 41st Streets. The only exceptions to this continuity of work in New York was during the World War, when he was especially helpful in directing large numbers of men in the construction of Government housing and of Army Cantonments at New London, Conn., Chillicothe, Ohio, and Camp Sherman, Illinois, as well as other works at Fortress Monroe, Virginia.

Mr. Carr died at his home in East Orange, N. J., on June 18, 1924. The cause of his death was blood poisoning from an injured hand, from which he had suffered a long time, and acidosis, a final illness of several weeks.

He was married on February 9, 1888, to Josie M. Butterfield, of Simsbury, Conn., who survives him, as do their six children, Mrs. W. A. Swain, Mrs.

* Memoir prepared by Verne LeRoy Havens, M. Am. Soc. C. E.

C. M. Wilson, Robert Carr, the Misses Laura and Lucy Carr, and Mrs. P. W. Tiemann, all of whom had successfully completed their college work before his death.

Mr. Carr was a member of the Congregational Church and of the Yale Club of New York.

He was widely known as a strict disciplinarian, which rôle was irksome to those few who are more interested in what they can get out of life than in what they can put into it. His discipline, however, was never exercised except where necessary; he was a stern and silent man to the crowd which hurried by, but most gentle to those who knew him. It was such as he who came out of rock-ribbed Puritan New England and who without the blare of trumpets or hope of great reward wrought the base on which our country stands to-day.

Perhaps the greatest incident of life is death; the greatest hope, that it ever be delayed in happy surroundings until the autumn of the years; and the greatest reward at such time be like that of Albert Carr who could well say: "I have fought a good fight, I have finished my course, I have kept the faith."

Mr. Carr was elected an Associate Member of the American Society of Civil Engineers on March 2, 1892, and a Member on October 7, 1903. His contributions to the *Transactions* of the Society included discussions on the Ninety-Sixth Street Power Station, New York, N. Y.,* and tuberculation in water pipes.†

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* *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 144.

† *Loc. cit.*, Vol. XXVIII (1893), p. 261.

ANDREW ARNOLD COHILL, M. Am. Soc. C. E.*

DIED JULY 2, 1924.

Andrew Arnold Cohill, the son of Andrew A. and Anastatia Egan Cohill, was born in Williamsport, Pa., on December 9, 1869. His father, who was a Civil Engineer, built most of the early bridge work of the Pennsylvania Railroad and was in charge of the old Pennsylvania Canal, which went out of existence in 1889, at the time of the Johnstown flood.

Mr. Cohill received his early education in the public schools of Williamsport, and, later, attended a private school where he studied engineering. On the completion of his course his father obtained for him a position with the Engineer Corps of the Pennsylvania Canal which was engaged in setting 1 000-yd. property monuments. He afterward served on the original survey of the Watsontown and Shickshinny Railroad from Watsontown to Shickshinny, Pa., which was known as the Watsontown and Western Railway. After spending considerable time in survey work, Mr. Cohill accepted a position at Altoona, Pa., in the Engineering Department of the Pennsylvania Railroad, Pittsburgh Division.

In 1890, many of the Pennsylvania Railroad officials transferred their services to a more recently established railroad—the Norfolk and Western. Mr. Cohill, who went with them, was made Assistant Supervisor at Pulaski, Va., and shortly afterward was given the position of Supervisor of the New River Division, between Radford, Va., and Bluefield, W. Va., with headquarters at Oakvale, W. Va. At this time the double tracking on this Division had begun, and it was noted by officials of the Norfolk and Western Railway Company that they were assigning their youngest Supervisor, not only in point of service, but in age, to the most important Division. Mr. Cohill executed the work of this Division ably throughout the entire period of double tracking.

Mr. Cohill then accepted the position of Engineer for Clark and Westbrook, Contractors, who were building a four-track road on the New York, New Haven and Hartford Railroad, at Mount Vernon, N. Y. This was heavy construction work, the length of the section extending from Woodlawn to New Rochelle, N. Y., a distance of approximately 5½ miles, and the tracks, which originally passed through Mount Vernon at street level, were depressed at some points as much as 30 ft. below the original grade.

Mr. Cohill later became associated with the Schoen Pressed Steel Car Company, at Pittsburgh, Pa., at its old Alleghany Plant. This was at the beginning of the pressed steel car industry. He was made Superintendent, but soon resigned to become Superintendent of the Keystone Bridge Works at the time the American Bridge Company was organized in 1900.

In 1902, Mr. Cohill was given the engineering as well as the construction work on the new furnaces for the National Tube Company at McKeesport, Pa. During a period of two years he built a new pumping plant and placed a new foundation under the furnace, keeping it in operation at the same time.

* Memoir prepared by Hunley Abbott, M. Am. Soc. C. E.

From 1904 to 1906, he served as Superintendent and Engineer on construction work for the American Telephone and Telegraph Company, building an underground system between Boston, Mass., and Providence, R. I., Trenton and Camden, N. J., and Racine and Milwaukee, Wis. From 1906 to 1908, he held the position of Superintendent and Engineer for S. Pearson and Son on the construction of the Pennsylvania Railroad Tunnels under the East River, New York, N. Y., and, from 1908 to 1909, he studied and traveled abroad.

Mr. Cohill was connected with T. A. Gillespie and Company from 1909 to 1912, serving one year on the construction of retaining walls along the Monongahela River, at Pittsburgh, Pa., and two years as General Superintendent on Contract No. 68 of the Catskill Aqueduct, including the construction of seven steel siphons, from Peekskill to Yonkers, N. Y. From 1912 to 1914, he held the position of General Superintendent for James Stewart and Company on the New York State Barge Canal Contracts 5-A, 59, and 72-A, and, in 1914, he joined the staff of P. McGovern and Company, with which Company he remained until 1920. He served for two years as Superintendent and Chief Engineer on the construction of the State Barge Canal Contract 71-A, and on the construction of Fort Point Channel, Boston, and for two years as Chief Engineer on the construction of Route 61 of the New York City Subways, comprising elevated, subway, compressed air, and hard rock tunnel work under the East River at Blackwells Island, and also on the construction of cylinder foundations for the Quartermaster's Terminal, at Boston.

In May, 1920, Mr. Cohill left the employ of P. McGovern and Company, and shortly afterward became associated with large financial interests in the preparation of a bid on a vehicular tunnel, at New York. This work engaged his attention for considerably more than a year. His estimate was carried out in the finest detail, but at the last minute his employers decided not to submit his bid. If they had done so, his estimate would have been slightly under those of all other bidders on this work and, undoubtedly, would have secured the contract.

During the last two years of Mr. Cohill's life, he served as General Manager for Abbott, Merkt and Company, Industrial Engineers, of New York.

He was not only an experienced Engineer, but particularly capable in handling construction work and directing men in the field. He quickly gained their confidence, and thereafter commanded them with all the precision of a military organization. Few men of his generation knew more than he concerning the theory and practice of difficult tunnel construction. His work on the foundations of the Quartermaster's Terminal in Boston, during the World War, set a record for speed in this type of work that probably never will be equalled.

Mr. Cohill was endowed with a most lovable personality, being a good friend to all with whom he came in contact and generous almost to a fault. He is survived by Mrs. Mercedes Good Cohill, to whom he was married at McKeesport, Pa., in October, 1903.

Mr. Cohill was elected a Member of the American Society of Civil Engineers on January 13, 1919.

ALBERT LADD COLBY, M. Am. Soc. C. E.*

DIED APRIL 30, 1924.

Albert Ladd Colby was born in New York, N. Y., on June 26, 1860, of John Ladd, M. D., and Mary Ann Colby. He was educated in the public schools of New York, at the College of the City of New York, and in 1881 received the degree of Bachelor of Philosophy from the Columbia School of Mines. Mr. Colby continued his studies at Columbia, acting occasionally as Assistant to Professor C. F. Chandler, until 1883.

For the next three years he was Assistant Professor of Chemistry at Lehigh University; then, in turn, Head Chemist and Metallurgical Engineer of the Bethlehem Steel Company, at Bethlehem, Pa., occupying the latter position until 1903. As Head Chemist, Mr. Colby's active and inventive mind devised several improvements in apparatus for increasing the rapidity and accuracy of the analytical work; as Metallurgical Engineer, he was, from time to time, in charge of the Blast Furnace, Open-Hearth, Bessemer, and Puddling Departments; he also had entire charge of the inspection, assignment, and economic use of all metallurgical materials, and he early gave his attention to the formulation of specifications for such materials—an activity in which he long held a distinguished part in the United States, and as its representative in England and Europe.

Mr. Colby also served as Expert for the Bethlehem Steel Company in patent suits, again laying the foundation for valuable service later as a Consulting Engineer. In this latter capacity, he was excelled by few, if any, contemporaries in the iron and steel industry. His mind was ingenious, thorough, clear, and keen, both in the preparation of evidence and its presentation to the Court. He very quickly appraised the value of evidence, or of an answer to a question, and knew how to secure the information, either from the available literature of any language, or by special researches or plant studies. In the executive control of literary research for patent purposes, his long training, infinite capacity for digging through long series of articles in English, German, or French, and his clear formulation of the vital subject-matter for the guidance of assistants, made him invaluable.

Mr. Colby was also an adept in collecting information on special branches of his profession. For example, when the science of Metallography began to assume importance, he made the first really comprehensive bibliography covering the literature of all siderurgical countries. For the newly-formed International Nickel Company, he gathered all the information extant on the subject of nickel steel, both in the literature and in the minds of experts in America and Europe. Two books testify to the thoroughness of his work in this capacity, namely "American Standard Specifications for Steel", 1902, and "Reinforced Concrete in Europe", 1909, besides many published monographs.

On the completion of his work for the International Nickel Company, Mr. Colby practiced as Consulting Engineer, with an office in New York, and,

* Memoir prepared by Bradley Stoughton, Esq., Bethlehem, Pa.

later, in Bethlehem also. He practiced a great deal in Europe, representing American firms and inventors there, for which his familiarity with the French and German languages and his wide acquaintance among the prominent iron and steel men especially fitted him. Although he was connected with the Bethlehem Steel Company, from 1886, he was in demand by other companies as Consulting Engineer or expert in patent suits. In 1906, he began to specialize in by-product coke manufacture, and practiced in this industry for several years. He was representing an American firm abroad, when he was stricken with influenza in London, and died at Torquay, England, on April 30, 1924.

On June 20, 1894, Mr. Colby was married to Agnes Wilson Lee, of Lewistown, Pa., who, with two children, survives him.

Mr. Colby will be remembered particularly for his life-long work on the specifications of steel, for his contributions to the advance of metallurgical knowledge in his several compilations of specialized information, for his public work as a representative of the industry, and for his active and disinterested support of technical societies.

In 1900, he represented the Bethlehem Steel Company at the Paris Exposition, and served as Juror in Metallurgy and Official Representative of the American Association of Steel Manufacturers; he was also United States Representative at the International Congress for Testing Materials of Construction. He was Special Iron and Steel Commissioner in the Department of Mines and Metallurgy at the St. Louis Exposition, in 1904, and, during the World War, took an active part in the Standardization of Aircraft Steels. He was Secretary of the American Association of Steel Manufacturers, which, during his incumbency (1897 to 1905), made the first successful effort to standardize specifications for finished steel, and he served actively on Committees of the American Association for Testing Materials, and its international associate, in formulating steel specifications. Space does not permit the listing of all the committees on which he served in his various technical societies, which included the Iron and Steel Institute of Great Britain, the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, the American Chemical Society, the Society of Chemical Industry, the American Iron and Steel Institute, the Franklin Institute, the German Iron and Steel Institute, the American Foundrymen's Association, and the Engineers' Society of Western Pennsylvania.

Mr. Colby was elected a Member of the American Society of Civil Engineers on October 7, 1903.

CHARLES EDWIN COLLINS, M. Am. Soc. C. E.*

DIED JANUARY 8, 1925.

Charles Edwin Collins, the son of Michael and Mary (Cushing) Collins, was born in Roxbury, Vt., on February 23, 1867. In 1870 his parents moved to Northfield, Vt. and he prepared for college in the public schools of that town. In 1886, he entered Norwich University at Northfield, from which he was graduated in 1890 with the degree of Bachelor of Science in Civil Engineering. While at Norwich, he was Corporal, Sergeant, and First Lieutenant and Adjutant in the Corps of Cadets, and a member of the Alpha Sigma Pi Fraternity. He received the degree of Civil Engineer in 1892.

Following his graduation, from 1890 to 1892, Mr. Collins was Assistant Engineer to the late James B. Francis, Past-President, Am. Soc. C. E., for the Locks and Canal Company at Lowell, Mass., and from 1892 to 1895, Principal Assistant Engineer for Pittsfield, Mass., in charge of extending water mains, of constructing reservoirs, surface drainage, sewers, and street pavements, and of surveys. From April to August, 1895, he served as Chief of Party on the construction of a sanitary sewerage system for Malden, Mass., and from 1895 to 1896, as Assistant Engineer in charge of the construction of Payson Park Reservoir for the City of Cambridge, Mass. In May, 1896, he was appointed City Engineer and Superintendent of Water-Works at St. Albans, Vt., which position he retained until March, 1897, when he was engaged as Assistant Engineer on the construction of overhead bridges on the Boston and Albany Railroad. In July, 1897, Mr. Collins became Assistant Engineer for the late Frank L. Fuller, M. Am. Soc. C. E., Consulting Engineer, of Boston, Mass., on the construction of sewers and water-works. In October, 1898, he entered the employ of William F. Shunk as Assistant Engineer in charge of surveys and the location of a railroad between Guayaquil and Quito, Ecuador, where he remained until May, 1899, at which time he was appointed Assistant Engineer with the late W. C. Johnson, M. Am. Soc. C. E., on the preparation of plans for a 10 500 h.p., hydro-electric power plant at Niagara Falls, N. Y. From May, 1900, to May, 1901, Mr. Collins was engaged with Alexander Potter, Assoc. M. Am. Soc. C. E., as Engineer in charge of the construction of sewerage and water-works systems, and from May, 1901, to July, 1903, as a member of Collins Brothers, Consulting Engineers, Philadelphia, Pa. During this time he designed and supervised the construction of several water and sewerage systems, power plants, etc.

From July, 1903, to his death on January 8, 1925, at Athens, W. Va., Mr. Collins was actively engaged as a Consulting Engineer on sanitary and hydraulic work, with offices in Philadelphia. During this time, he constructed sewerage systems and sewage disposal plants at Lake Placid, Dolgeville, Granville, Malone, and Ticonderoga, N. Y.; Bedford, Pottstown, and Danville, Pa.; Shelbyville, Mt. Sterling, Versailles, and Winchester, Ky.; Morristown, Tenn.; and Moundsville and Weston, W. Va.; as well as water-works at Milton, Del., and Bridgeton, N. J. The reinforced concrete reser-

* Memoir prepared by John L. Collins, Esq., New York, N. Y.

voir at Bridgeton, when completed, was the largest of its kind in the United States. Mr. Collins had also constructed water supply and pumping plants at Lake Placid and Ticonderoga, N. Y., and at Versailles, Ky., and electric light and power plants at Malone, Chateaugay, and Schaghticoke, N. Y.

For five years he had also served as Consulting Engineer for the State Board of Control for West Virginia.

Mr. Collins was a member of the Chestnut Street Baptist Church, Philadelphia, Pa., and of DeWitt Clinton Lodge, A. F. and A. M. of Northfield, Vt. He was also a member of the Engineers Society of Pennsylvania.

He was married on October 8, 1901, to Mabel May Tuggey, of Pittsfield, Mass., who, with two sisters, Dr. Emma H. Collins and Dr. Alice L. Collins, both of Philadelphia, and two brothers, J. Henry Collins, of Lowell, Mass., and John L. Collins, of New York, N. Y., survives him.

Mr. Collins was elected an Associate Member of the American Society of Civil Engineers on March 2, 1904, and a Member on September 13, 1913.

ARTHUR BATEMAN CORTHELL, M. Am. Soc. C. E.*

DIED MAY 24, 1924.

Arthur Bateman Corthell, the son of James H. and Charlotte Almy Corthell, was born on July 3, 1860, in Whitman, Mass. His father was a veteran of the Civil War.

Mr. Corthell was educated in the public schools of Providence, R. I., and entered Brown University in 1877, completing his course in March, 1881. At that time, Brown University had no comprehensive Engineering Course, but, in 1898, by a special vote of the Trustees of the University, Mr. Corthell was awarded the degree of Master of Arts.

During his college course, he was employed on vacations, and at other times, as a Draftsman for the Herreshoff Manufacturing Company of Bristol, R. I. As a result of his work for that company on the design and construction of yachts, he became an enthusiastic yachtsman and throughout his life owned or sailed yachts whenever an opportunity presented itself.

Following his graduation from Brown University, Mr. Corthell was employed for a few months as a Rodman in the City Engineer's office in Providence. In September, 1881, he left that position to take up railroad engineering as a profession, to which work he devoted the remainder of his life.

His first work was with the New York, West Shore and Buffalo Railroad Company, where he was employed from September, 1881, to December, 1884, as Assistant Engineer on Construction between Little Falls and Pittsford, as Supervisor on Maintenance, and, during the last year, as Division Engineer of that part of the line between Syracuse and Buffalo, N. Y.

Between December, 1884, and April, 1886, Mr. Corthell worked with the Knickerbocker Ice Company of New York, N. Y., making surveys and locations for the Rockland Lake Railroad and on miscellaneous engineering projects. He returned to the employ of the New York, West Shore and Buffalo Railroad Company early in 1886 and was engaged as Draftsman in the office of the Chief Engineer until the middle of that year.

In September, 1886, he entered the employ of the Fitzgerald and Mallory Construction Company and, as Principal Assistant Engineer, worked on the construction of about 300 miles of new railroads in Kansas and Colorado for the Missouri Pacific System.

From July, 1887, to March, 1888, he was employed as First Assistant Engineer on the construction of the Sioux City Bridge, over the Missouri River, this work carrying with it heavy pneumatic foundations with piers sunk to a depth of 92 ft.

In 1888 and 1889, Mr. Corthell served as Assistant Engineer on the construction of the Thames River Bridge and its approaches, at New London, Conn., having charge of the construction of the eastern half of the bridge foundations and the eastern approach, about 5 miles long, re-locating and designing the masonry structures.

* Memoir prepared by F. C. Shepherd, M. Am. Soc. C. E.

In 1889, he became Assistant Engineer of the New York, Providence and Boston Railroad, on design and construction work and, in 1892, he was made First Assistant Engineer in charge of the design and construction of the Providence Terminal, with its approaches, being associated in this work with the late George B. Francis, M. Am. Soc. C. E.

After the completion of the Providence Terminal, Mr. Corthell followed Mr. Francis to Boston, Mass., and was employed under him from 1897 to 1899 as Principal Assistant Engineer on the construction of the South Station, one of the largest passenger terminals in the world.

On the completion of this station, he was engaged for a few months with Westinghouse, Church, Kerr and Company in Boston, returning, in February, 1900, to the South Station, where for two years he was Resident Engineer for the Boston Terminal Company, leaving there in February, 1902, to accept a position with the New York Central Railroad.

From 1902, to July, 1911, Mr. Corthell was with the New York Central Railroad, as Terminal Engineer for four years; as Assistant Executive to the Grand Central Station Architects for two years; and for the last three years as Secretary and Consulting Engineer of the Auxiliary Facilities Committee of the Grand Central Terminal.

From his association with Mr. Francis and his experience at Providence and Boston, Mr. Corthell became known as one of the leading experts in the country on terminal developments, and this undoubtedly largely influenced his selection for the position mentioned.

Under his direct supervision, the original plans of the Grand Central Terminal improvements were developed, and he was largely instrumental, not only in the preparation of the final plans, but also in the carrying out of the work toward its completion. It has been said of his relations with many associates in the planning and construction of this great terminal, that it was largely due to his efforts that the work went on harmoniously and without delay.

In 1911, Mr. Corthell became Chief Engineer of the Boston and Maine Railroad, being selected for this position because of the large construction plans contemplated, at that time, a number of which have been carried out. This position he filled until his sudden death while spending the week-end at his farm in New Boston, N. H.

He is survived by his wife, Mrs. Lena Foster Corthell, a daughter, Mrs. John T. Phelps, and two sons, Mark A. and Edwin L. Corthell, as well as by his father and mother.

As a boy, Mr. Corthell was a member of the choir of Trinity Church, Bristol, R. I., and always, during his life, was a great lover of music. He was a member of the Protestant Episcopal Church and at the time of his death had been a Vestryman of the Church of the Epiphany, in Winchester, Mass., for several years.

He was an enthusiastic Mason, having been affiliated with Orpheus Lodge, of Providence, R. I. He was also a member of the Massachusetts Consistory.

Mr. Corthell was a man of unusual character and integrity, had a very quick, keen mind, and his ability was fully recognized in his profession, as well as by the railroad executives under whom he was employed. He was a man of many friends and one who was greatly beloved by all who ever became associated with him. His death is a distinct loss to the Engineering Profession.

He was a member of the Boston Society of Civil Engineers, the American Railway Engineering Association, the New England Railroad Club, and the Vesper Country Club of Lowell, Mass.

Mr. Corthell was elected a Member of the American Society of Civil Engineers on March 1, 1899.

EDWARD BENJAMIN CUSHING, M. Am. Soc. C. E.*

DIED FEBRUARY 17, 1924.

Edward Benjamin Cushing was born in Houston, Tex., on November 23, 1862. He received his first instruction in the public schools of Houston and because of his rare aptitude for mathematics, advanced with unusual rapidity. He was afterward sent to the Texas Military Institute at Austin, where he remained until the opening of the Agricultural and Mechanical College of Texas, at Bryan, in 1877. He remained at this institution during 1877 and 1878, a member of the first engineering class, intending to complete his course, but his work was interrupted by the death of his father, the late E. H. Cushing, a prominent writer and newspaper owner of Houston. He returned to the college later to resume his studies and, in 1899, received the degree of Civil Engineer.

Mr. Cushing obtained his first practical training in engineering under Col. Ashbel Smith, a noted scholar and at one time Minister Plenipotentiary and Ambassador Extraordinary from the Republic of Texas to the Court of St. James. While engaged with Colonel Smith, Mr. Cushing located the boundaries that marked many a league, including territory covering the shores of Trinity Bay and the Gulf of Mexico, around the San Jacinto Battle-Grounds, and Goose Creek, the richest oil fields in this section of the State. Many of these lines are recognized to-day.

Mr. Cushing then served as an Axeman under the late James P. Converse, Chief Engineer of that branch of the present Southern Pacific Lines, then known as the Galveston, Harrisburg and San Antonio Railway. Later, he was promoted to the position of Chief of Party and located a line from San Antonio, Tex., into Mexico. This work was accompanied by the hard knocks and hazards characteristic of a rough frontier country. In 1881, he was advanced to the position of Engineer in Charge of Construction on the Galveston, Harrisburg and San Antonio Railway.

In 1882, Mr. Cushing was engaged as Assistant Engineer on the location of a line from Piedras Negras to Paila, Mexico, a distance of 200 miles, following which, during construction, he was made Assistant Engineer of Track. In 1883, he accepted a position as Assistant Engineer with the Galveston, Harrisburg and San Antonio Railway Company, and was in charge of re-alignment and grade reduction, as well as yard remodeling. During 1885 and 1886, he served as Assistant Engineer in charge of drafting and accounting for the Gulf, Colorado and Santa Fé Railway Company, during much construction work in Texas and Indian Territory. Later, he was promoted to be Assistant Engineer in Charge of Roadway, Bridges, and Buildings.

In 1889, Mr. Cushing returned to the employ of the Galveston, Harrisburg and San Antonio Railway Company, which had been merged with the Southern Pacific Company, as Resident Engineer in Charge of Track and Roadway, which title he retained until 1897. During this time, he was assigned to the

* Memoir prepared by H. F. Jonas, M. Am. Soc. C. E.

rehabilitation of the Houston East and West Texas Railway, extending from Houston, Tex., to Shreveport, La. Later, he was appointed Chief Engineer and General Superintendent of this Company with which he remained for five years.

He was then transferred to the Southern Pacific Lines as Engineer on Maintenance of Way of the Atlantic System, with jurisdiction from New Orleans, La., to El Paso, Tex., including the Houston and Texas Central Railroad—a total mileage of 3 200. In 1916, Mr. Cushing was appointed Assistant General Manager of the Southern Pacific Lines, which position he held until he left for overseas service.

In June, 1917, he received an appointment as Major of the 7th Regiment of Engineers, later the 17th Regiment, which was mustered into service at Atlanta, Ga. The regiment sailed immediately for France, and, after reaching Havre, Major Cushing was transferred to the Railway Transportation Corps. His territory included La Pallice, France, a very important point in the operation of the Expeditionary Forces. On May 25, 1918, he was made Chief Transportation Officer of Base Section No. 6 of the American Expeditionary Forces. In September of the same year, he was promoted to the rank of Lieutenant Colonel in the Transportation Corps, and, in October, he was appointed General Superintendent of the Army Transportation Corps, covering Mediterranean ports. He received distinguished service medals from the Governments of France, Belgium, and Italy, from the General-in-Chief of the American Forces, and the French Legion of Honor.

Colonel Cushing entered the Army with the rank of Major, through test examinations and strong endorsements, although he offered to serve as a Lieutenant rather than miss the opportunity of service abroad. To do so he resigned from an official railroad position that paid him a higher salary than any military position offered.

On his return from France, he resumed his position as Assistant General Manager with the Southern Pacific Lines, Texas and Louisiana, but remained only a short time, having been retired after forty years of service on account of physical disability.

Colonel Cushing was elected to honorary membership in the Engineering Association of the South, and received a similar compliment from the Franklin Institute, Philadelphia, Pa. He was a Thirty-second Degree Mason and a member of the Houston Lodge of the Benevolent Order of Elks.

Among other military honors conferred on Colonel Cushing, were appointments by Governors of two States—Texas and Louisiana—to the rank of Lieutenant-Colonel in crack Guard regiments of those States. In Texas, he was called into active service with the State Guard on more than one occasion, responding each time on short notice and acquitting himself with credit.

He acted as Chairman of the Board of Directors of the Agricultural and Mechanical College of Texas, at a time when the college was temporarily without a President, and until the present head of the College, Dr. W. B. Bizzell, was appointed. Colonel Cushing also served as President of the Alumni Association.

Soon after his retirement from active railroad service, he was appointed Receiver of the Ranger National Bank, which institution grew to mammoth proportions during the boom days of the Ranger Oil Field, but collapsed under the strain of deflation. Colonel Cushing, held this position at the time of his death in Houston, on February 17, 1924.

He is survived by a son, James Converse Cushing, two married daughters, one brother, four grandchildren, and a host of loyal friends to mourn his untimely death.

Colonel Cushing was elected a Member of the American Society of Civil Engineers on November 1, 1893.

THOMAS REMINGTON HOLDEN DANIELS, M. Am. Soc. C. E.*

DIED NOVEMBER 7, 1923.

Thomas Remington Holden Daniels, the son of George Metcalf and Ellen Eliza (Holden) Daniels, was born in Providence, R. I., on February 19, 1876. His father died when he was nine years of age, so that his education was dependent on his own efforts. He was graduated from High School at the age of sixteen, and then went to work, improving his education by the study of civil engineering at night schools. The greater part of his technical knowledge however was gained by practical experience.

From 1892 to 1896, Mr. Daniels was a student in the office of R. H. Tingley, Consulting Civil Engineer, of Providence, R. I., and from 1897 to 1898, he was with J. A. Latham, Civil Engineer, also of Providence.

In 1898, Mr. Daniels entered the electric railway field as Assistant Engineer in charge of track and roadway of the Rhode Island Company, Providence, R. I. He left that position in 1904 to become Assistant Engineer for Westinghouse, Church, Kerr and Company, on the construction of the new Terminal Station for the Pennsylvania Railroad Company, in New York, N. Y. During his connection with this Company, he was also engaged on preliminary surveys for the high-tension line from Rochester to Syracuse, N. Y. He resigned from the Westinghouse, Church, Kerr and Company in 1906 and took charge as Assistant Engineer of the rebuilding of tracks for the United Railroads of San Francisco after the earthquake. In this capacity he reconstructed some thirty miles of old cable road and also had charge of the installation of special work.

In 1908, Mr. Daniels severed his connection with the United Railroads of San Francisco to become Chief Engineer of Maintenance and Construction for the Terre Haute, Indianapolis and Eastern Traction Company, and as such he assisted in the design and construction of the West Tenth Street Power Plant at Indianapolis, Ind. He left this Company in 1911 to go to Birmingham, Ala., as Chief Engineer of the Birmingham Railway, Light and Power Company. In addition to street railway work, he also had charge of the Power and Gas Division. In 1914, he returned to Indianapolis as Chief Engineer for the Terre Haute, Indianapolis and Eastern Traction Company, in charge of all track work, bridges, and buildings.

In 1911, Mr. Daniels was married to Effie Cade, of Baton Rouge, La., who survives him. He died November 7, 1923, at his Indianapolis residence and was interred at Baton Rouge, La.

He was a man of modest disposition, high ideals, and absolute integrity, and was very conscientious in his work. He was a great lover of good music, being himself a capable pipe organist. He was affiliated with the American Association of Engineers, and was a Scottish Rite and a York Rite Mason, as well as a member of the Shrine.

Mr. Daniels was elected an Associate Member of the American Society of Civil Engineers on February 6, 1907, and a Member on June 19, 1918.

* Memoir prepared by T. H. David, M. Am. Soc. C. E.

JUSTUS VINTON DART, M. Am. Soc. C. E.*

DIED SEPTEMBER 26, 1923.

Justus Vinton Dart, the son of Eben E. and Juliet (Hurlbut) Dart, was born in New London, Conn., on October 11, 1849. His ancestral line probably leads back to Richard Dart, who is recorded as having been a resident of New London in 1664 and the head of a large family.

Mr. Dart's education, begun in the public schools of his native city, was completed at the Department of Yale University now known as the Sheffield Scientific School, from which he was graduated in the Class of 1871.

In May, 1872, he entered the office of Charles E. Paine, City Engineer of Providence, R. I., and for some years was engaged in the various lines of municipal work. Much of his time, however, was devoted to surveys, plans, and estimates for the construction of new streets required by the rapidly growing city; in 1877, the newly elected City Engineer, the late Samuel M. Gray, M. Am. Soc. C. E., in re-organizing the office, placed Mr. Dart in charge of that work. Later, in 1881, he became the head of the Highway Department of the City Engineer's Office, his duties including the design and adjustment of street grades, the preparations of plans, estimates, and specifications for street building and paving, the supervision of highway work in progress, and the location of street railways.

During Mr. Dart's term of service for the city, important changes were taking place. In area, the municipality gained approximately 100%; in population, the increase was, perhaps, two and one-half times greater. Generally speaking, highway travel and traffic were tremendously stimulated by the invention of new methods of locomotion and the urgent need for smoothly surfaced streets with ample stability for the transit of heavier loads inspired a corresponding activity in the invention of new methods and the use of new materials for highway construction.

In the early Seventies, the more important streets of Providence were paved with cobblestones laid on a sand foundation, while those subject to lighter traffic were surfaced with screened gravel. The use of the steam roller for consolidating the foundation courses and finishing the wearing surfaces had only recently begun. During this decade, the matter of improved street surfaces received much attention, and the use of granite blocks laid on a sand or gravel foundation came into vogue. Some rather unsuccessful experiments with wooden blocks and briquettes of crushed limestone and asphalt were also undertaken. A careful study was made in preparing for the improvement of Westminster Street—an important thoroughfare in the business section—and, in 1880, the work was performed under the supervision of Mr. Dart. It was later stated by him to have been the "first permanent pavement constructed by the City of Providence".

The specification called for a 5-in. foundation course of natural cement concrete, on which a bed of clean, dry gravel, 1½ in. thick, was evenly spread,

* Memoir prepared by Herbert E. Sherman, M. Am. Soc. C. E.

to receive granite blocks, laid with open joints. The joints were then filled with clean, hard, dry sand—artificially dried if necessary—and the blocks rammed to an even surface, after which the joints were saturated with a heated mixture of coal-tar pitch and creosote oil. This pavement stood up under heavy traffic for more than 40 years, being replaced by sheet asphalt in 1922, when the underlying foundation was found to be intact and to require only the additional material for bringing the thinner asphalt surfacing to the established grade.

From this time, the use of improved methods, gradually introduced, was steadily followed. At the close of Mr. Dart's connection with the Highway Department, the City had about 260 miles of municipal streets of which, about 70 miles were paved in accordance with modern practice and less than 3 miles were of cobblestone construction. Of newly accepted streets, 8 miles remained unbuilt, and the residue, located chiefly in the outer districts, were of water-bound or bituminous macadam.

In the spring of 1915, Mr. Dart was appointed Superintendent of Highways, assuming the charge and daily inspection of the outside street construction, in addition to the direction of his Department in the office of the City Engineer. He resigned from the employ of the city on July 1, 1916, and resided in Thompson, Conn., until his death.

In a review of Mr. Dart's professional career, one may consider him a type of the great army of municipal engineers, who steadily labor, year after year, in the interest of the public. Earnest, honest, industrious, intelligent, and capable, their virtues are not always appreciated. The merest details of their minor duties call for painstaking care and accuracy, while trained judgment and scrupulous impartiality are no less required in matters of greater importance.

Skilled in his vocation, diligent in the prosecution of business, Mr. Dart, was courteous and agreeable to his associates, fair and considerate to his subordinates, and generous and loyal to his many friends.

He was married on January 1, 1894, in New London, to Alice F. Howard, who survives him.

Mr. Dart was elected a Member of the American Society of Civil Engineers on April 4, 1900.

HENRY HARRISON FARNUM, M. Am. Soc. C. E.*

DIED OCTOBER 23, 1924.

When Henry Harrison Farnum at the age of eighty passed from earth on October 23, 1924, he left behind him but three members of his class of 1865 and perhaps not more than sixteen or seventeen of Rensselaer's Alumni who had been graduated before that time. There are, therefore, few to tell the history of those early years.

Mr. Farnum's paternal ancestor, Henry Farnham, came from Old England to New England in 1638. His mother's ancestor, Jacob Caudebec, of French Huguenot stock, left his home, in Caudebec, France, shortly after the Revocation of the Edict of Nantes, reaching a permanent home in New York State about 1690. The subject of this brief sketch may well be called an American.

Many representatives of the family had chosen the profession which Mr. Farnum made his own, and it was, therefore, quite in keeping that in 1865, at the age of twenty-one, he should have completed the course in Civil Engineering at Rensselaer Polytechnic Institute, Troy, N. Y., entitling him to the degree of Civil Engineer. While at Rensselaer he became one of the founders of the Theta Xi Fraternity, a strictly engineering organization with twenty-seven Chapters located at engineering schools and universities having engineering courses. In this fraternity he took a hearty interest and was always proud of his connection as one of its founders.

His boyhood and early youth were spent in Port Jervis, N. Y., where one of the lineal descendants of Jacob Cuddeback—Asenath Cuddeback—had become the wife of Samuel B. Farnum, a descendant in direct line from Henry Farnham, both family names being Americanized. Their fourth child was Henry Harrison. At the age of fourteen, he was sent to Flushing Institute, then presided over by Elias Fairchild, of whom his former pupil could never speak too highly.

So few remain who knew Mr. Farnum during his life in Troy that it seems not inappropriate to quote a few sentences penned by one who was his friend there, and whose friendship has continued to this day. "A most lovable friend," the letter says, "kind and sympathetic. A fine scholar. His sympathy included all who needed his help and it was always cheerfully given."

The years from 1866 to 1868 were spent in mining engineering in Colorado. He crossed the plains before the Union Pacific had made its way over, and he sometimes spoke of that journey of eight days and nights in a Concord coach with Indians lurking near enough at hand to make both driver and passengers continually alert.

After his return from Colorado, Mr. Farnum became Assistant Engineer on the preliminary surveys and location of the Port Jervis and Monticello Railroad in New York. During 1869-70, he served as Assistant Engineer for the New Orleans, Mobile and Chattanooga Railroad, engaged in surveys for location and construction, especially for the westerly extension.

* Memoir compiled under the direction of the late William H. Wiley, M. Am. Soc. C. E.

From 1870 to 1875, Mr. Farnum was Chief Engineer for the South Side Railroad of Long Island. He lived then in New York, or in Brooklyn, N. Y., and his days were given to engineering, his evenings to the studious enjoyment of the best that New York could offer to satisfy his ardent love for the drama and music, particularly the latter. Theodore Thomas was then a great figure in the musical life of the city, and the young engineer derived infinite pleasure and profit from the gratification of his fine taste for the best.

After finishing his work with the South Side Railroad, private and miscellaneous engineering engaged Mr. Farnum's attention until 1881, and he was for several years engaged with experimental investigations in the process of refining iron, embracing the designing and construction of an iron plant at Chicago, Ill., including heavy machinery and a very heavy foundation.

In 1890, he returned to New York and entered the service of the City in what is now the Borough of the Bronx, and, from that time, he was engaged almost continuously in this Borough, first as Assistant Engineer, then as Chief Engineer of Sewers. At the time of his retirement from active service, about 1923, after many years of profound and intelligent interest in the phenomenal growth and development of the Borough, he was still occupied in the exercise of his professional skill toward making that development healthful.

The period from 1890 to 1901 covers the development of the sewer system in this Borough. It was here Mr. Farnum's fortune to build the largest and longest sewer in the United States. Owing to the varying natural conditions of the drainage distribution of the extensive marsh lands bordering on the Harlem River, the East River, and Long Island Sound, new problems were presented in the case of each outlet sewer, in order to secure a permanent foundation. This task was successfully accomplished.

The closing years of Mr. Farnum's earthly life were greatly enriched in affording more leisure for historical reading which was always his delight, in giving time for home pursuits which were always dear to him, and in following with great interest the developing lives of the younger members of the family. There were many on both sides of the house who bear joyful witness to the stimulus of his influence.

He was married, in 1887, to Elizabeth Beattie, who, with his son, Dr. Waldo B. Farnum, his daughter-in-law and two grandchildren, survives him.

Mr. Farnum was a member of the Presbyterian Church from early manhood. As son, brother, husband, and father, in all the most intimate relationships as well as in those more widely known, his life has spoken, and those who knew him in each phase of his long and varied career will bear glad tribute to his worth. One who knew and cherished him wrote, "A gentleman, gallant and unafraid, has passed on." Permit one added word, "A Christian gentleman, gallant and unafraid, has passed on."

Mr. Farnum was elected a Member of the American Society of Civil Engineers on July 1, 1891.

CHARLES ADDISON FERRY, M. Am. Soc. C. E.*

DIED JULY 31, 1924.

Charles Addison Ferry was born at Granby, Conn., on January 8, 1852. In 1871, he was graduated from the Civil Engineering Department of the Sheffield Scientific School of Yale University, with the degree of Bachelor of Philosophy. In 1891, he received the degree of Civil Engineer.

Soon after his graduation, Mr. Ferry entered the office of the City Engineer of New Haven, Conn., and was assigned to charge of a party making a survey and map of the city. In a short time he was promoted and placed in special charge of sewer construction and all work in connection with stone arches, bridge abutments, retaining walls, etc., which position he retained until the early part of 1892.

From May, 1892, to May, 1913, Mr. Ferry served as Principal Assistant in the office of A. B. Hill, M. Am. Soc. C. E., Consulting Engineer, of New Haven, his principal work being on the design and construction of water-works, sewerage systems, and the determination of property lines. Mr. Ferry had much to do with planning the details of the water-works systems of New Haven and neighboring cities.

From May, 1913, until his death, he maintained a private office for the general practice of civil engineering. He designed the Yale "Bowl" and was Resident Engineer on its construction. This beautiful structure will stand as a monument to his genius.

Mr. Ferry is survived by his wife and one daughter, Ruth M. Ferry.

He was a man of exceptionally clear, sound judgment, thoroughly reliable, and highly esteemed by those who knew him. His family life was ideal.

Mr. Ferry was a member of the New Haven Chamber of Commerce. He also took an active, responsible interest in the affairs of the Plymouth Congregational Church of New Haven, and had much to do with the planning of its new edifice.

He was a Past-President of the Connecticut Society of Civil Engineers and contributed papers on various subjects, including the Yale Bowl, the filtration plant of the New Haven Water Company, and the reinforced concrete ammonia tank of the New Haven Gas Company, to its publications. In 1916, he also presented a paper entitled "The Yale Bowl"† before the Society.

Mr. Ferry was elected a Junior of the American Society of Civil Engineers on May 4, 1881, and a Member on January 2, 1889.

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* Memoir prepared by Albert B. Hill, M. Am. Soc. C. E.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 249.

RUDOLPH FINK, M. Am. Soc. C. E.*

DIED FEBRUARY 1, 1913.

Rudolph Fink was born on November 29, 1834, in Lauterbach, Hesse-Darmstadt, Germany. He was the youngest of a family of twelve children born to Andreas and Margaret (Jacob) Fink. His father, being an architect, fully appreciated the advantages of a technical education and accordingly was able to give his sons such advantages. Rudolph attended the Polytechnic School in Darmstadt, about sixty miles from Lauterbach, from which he was graduated.

Prior to the completion of his special studies (about 1856), he had determined to come to the United States, where his elder brother, the late Albert Fink, M. Am. Soc. C. E., had been since 1849.

His first position in America was that of Assistant Engineer on the construction of the Baltimore and Ohio Railroad. In the spring of 1857, he was appointed Resident Engineer in charge of track-laying on the Norfolk and Petersburg Railroad on which road his brother, Henry, was engaged. Later, about 1858, Mr. Fink was induced by his brother Albert, then with the Louisville and Nashville Railroad Company, to go to Kentucky where, as a subordinate to his brother, he rapidly adapted himself to American methods of railroad construction. He served with the Federal Army in the Civil War with the rank of Major, and during that stormy period when the Louisville and Nashville Railroad was subjected to sudden and severe destruction, he devoted most of his time to repairing the damage done by the Confederate forces. This enabled him to become thoroughly familiar with all kinds of structures relating to bridges, buildings, and the maintenance of track, both temporary and permanent.

In 1865, when his brother, Albert, became General Superintendent of the Louisville and Nashville Railroad, Mr. Fink was promoted to the position of Engineer and Superintendent of the Road Department. He continued to hold this office until the early part of 1872.

About 1870, the process of converting scrap-iron rails into new ones by re-rolling was carried on by a number of iron works. About 1872, Mr. Fink left the Louisville and Nashville Railroad Company to become one of the proprietors of the Ohio Falls Iron Works, at New Albany, Ind., which company was then engaged in this process of manufacturing new iron rails. The company, however, was not successful, due to the gradual substitution of steel for iron and also due to the disturbance following the financial panic of 1873. Mr. Fink retired from this venture during the latter part of 1875 and accepted the position of Senior Assistant City Engineer of Louisville, Ky., under the late R. T. Scowden, M. Am. Soc. C. E., City Engineer, remaining in this position for nearly two years. In February, 1878, he was appointed Assistant and Superintendent of the Washington City, Virginia Midland, and Great Southern

* Memoir compiled under the direction of W. L. Mapother, Pres., Louisville & Nashville R. R. Co., Louisville, Ky., from data received from various sources.

Railroad, leaving in December of the same year to become General Manager of the Memphis and Little Rock Railroad.

In August, 1880, Mr. Fink accepted a position as General Manager of the Selma, Rome and Dalton Railroad, where he remained until October 1, when he moved to the City of Mexico and entered on the important duties of Chief Engineer on the construction of the Mexican Central Railroad which he completed several years later with great credit to himself. In this position, it was necessary for him to learn to speak and write Spanish in order to transact business with President Diaz and other officials in Mexico. This he readily accomplished as he was quite a linguist, speaking also, German, French, and English.

In April, 1884, after the completion of his work in Mexico, Mr. Fink became connected with the Memphis and Little Rock Railroad Company, with headquarters at Little Rock, Ark. A few years later, this company was re-organized and its name changed to the Little Rock and Memphis Railroad Company of which Mr. Fink became the President and General Manager, with headquarters in Memphis, Tenn., to which city he moved. After the merging of the Little Rock and Memphis Railroad into another system, Mr. Fink retired from all railroad duties and made Louisville his permanent residence until his death on February 1, 1913.

On November 7, 1872, he was married to Cary Anne Nicholas, the daughter of the late S. S. Nicholas. Five children were born to them, one son and four daughters.

Mr. Fink was elected a Member of the American Society of Civil Engineers on September 21, 1870.

WALTER JOSEPH FRANCIS, M. Am. Soc. C. E.*

DIED MARCH 6, 1924.

Walter Joseph Francis, the son of Joseph and Elizabeth Francis, was born in Toronto, Ont., Canada, on January 28, 1872. He received his education in the public schools of Ontario, the Toronto Collegiate Institute, and the University of Toronto, having been an Honor Graduate of the latter in 1893, with the University degree of Civil Engineer.

After his graduation, Mr. Francis became Assistant to the Chief Engineer in charge of the design of the Toronto Union Station. In 1896, he was appointed Chief Draftsman on Bridge Construction for the Central Bridge Engineering Company, but, at the end of a year's service, he entered the Department of Railways and Canals for Canada, on the works of the Trent Canal, in special charge of the design and construction of the two hydraulic lift locks at Peterborough and Kirkfield, Ont., respectively. For a paper on the subject of these lift locks read by him before the Engineering Institute of Canada, he was awarded the Gzowski Medal in 1906. At this time, he had charge of the 32 000-h. p. electric plant on the Kootenay River, at Bonnington Falls, B. C.

In 1907, Mr. Francis became Assistant Manager and Chief Engineer of the Dominion Engineering and Construction Company, at Montreal, Que., as well as Engineer to the Royal Commission on the Quebec Bridge Disaster. In 1908, he was appointed exclusive writer for *The Engineer*, published in London, England. In 1909, he reported on various hydro-electric power propositions, including works at Campbellford, Ont., and at Edmonton, Alberta, on the North Saskatchewan River. In 1910, he reported on the public utilities of the City of Edmonton; the Herald Building disaster and the Boxer Building collapse, at Montreal; the construction of the Don Siphon for main sewers for the City of Toronto; and the hydro-electric and steam plants for the City of Quebec, Que. He also represented the Canadian Society of Civil Engineers on the Committee appointed to revise the By-Laws of the City of Montreal.

In the same year (1910), Mr. Francis formed a partnership with Mr. Frederick B. Brown, which continued to the time of his death. This firm has had an active career, its members having reported on and designed numerous hydro-electric and steam plants, water supplies, sewage, and other municipal undertakings in some of the large Canadian cities. The firm was a member of the Water Board of Montreal, having been appointed at the time of its organization, and continued as such until charge of the work was taken over by the City Staff. In 1922, the firm was appointed as Consulting Engineer to the Royal Commission and charged by the Government of Ontario to investigate and make a complete report on the activities of the Hydro-Electric Power Commission of Ontario.

* Memoir prepared by J. M. R. Fairbairn, M. Am. Soc. C. E.

At the time of his death, Mr. Francis was President of the Engineering Institute of Canada, in which he had occupied a prominent position for many years, having been a most active and untiring worker in its cause.

He was Vice-President of the Corporation of Professional Engineers of Quebec in 1920; President of the Engineering Alumni Association of the University of Toronto; held membership in the Engineering Institute of Canada, the Institution of Civil Engineers of Great Britain, American Institute of Consulting Engineers (Charter Member), the Engineering Society of Toronto University (Life Member), American Public Health Association, Montreal Board of Trade, the University Club of Montreal, the National Club of Toronto, the Royal Societies Club, of London, England, and numerous other organizations. He held an appointment in the Rotary International for seven years, and was a Past-President of the Rotary Club of Montreal, as well as a member of several International Committees. He was also active in the Boy Scout and Wolf Cub movements, occupying the office of President of the Montreal Boy Scouts at the time of his death. He found time, in addition, for hospital work in two different institutions, as well as serving as Secretary of the Shriners' Hospital for Crippled Children. He was a Thirty-second Degree Free Mason, a member of the Royal Order of Scotland, and a Past Grand First Principal of the Grand Chapter of Royal Arch Masons of Quebec.

On November 26, 1896, Mr. Francis was married to Laura Elizabeth Grainger, of Toronto. He is survived by Mrs. Francis and two sons, Edward and Francis.

He was one of the outstanding engineers in the Dominion of Canada, and took an active and prominent part in every phase of the development of the Engineering Profession in the Dominion of recent years.

Mr. Francis was elected an Associate Member of the American Society of Civil Engineers on May 1, 1901, and a Member on April 5, 1904.

LEWIS SINGER GILLETTE, M. Am. Soc. C. E.*

DIED MARCH 30, 1924.

Lewis Singer Gillette was born at Niles, Mich., on May 9, 1854. He was the descendant of a pioneer New England family founded by Jonathan Gillette who came to America in the *Mary and John* and settled in Dorchester, Mass. Subsequently, members of the family moved to New York, N. Y., and, later, to Niles, Mich.

Mr. Gillette obtained his elementary education in the country school and High School at Niles. In 1872, he went to Minneapolis, Minn., where he entered the University of Minnesota from which he was graduated in 1876 with the degree of Bachelor in Civil Engineering; later, he received the degree of Civil Engineer from the same institution.

After graduation, Mr. Gillette returned to Niles and for two years was engaged in farming. In 1878, he bought the Niles Chilled Plow Works which he operated until 1882, when he returned to Minneapolis to cast his fortunes with that rapidly growing city.

Mr. Gillette was engaged as Right-of-Way Agent for the Hill railroads in 1882, and, in 1883, he accepted the position of Engineer for the St. Anthony Water Power Company. In 1884, he bought a half interest in the Herzog Iron Works which in 1889 became the Gillette Herzog Manufacturing Company of which he was made President. As head of this institution, he was an important factor in the development of the structural steel business in the West.

In 1899, the American Bridge Company was organized and took over the plant of the Gillette Herzog Manufacturing Company. Mr. Gillette continued for a short time with the American Bridge Company, in charge of all sales west of the Mississippi River, but his growing interests in other lines demanded his full attention; he therefore severed relations with that Company in 1900, and from then until his death devoted his time and energy to numerous enterprises in the organization of which he was instrumental. Among these undertakings was the Minneapolis Steel and Machinery Company of which he was one of the founders in 1902. This Company continued the structural steel business in the West so well begun by Mr. Gillette, and he retained his interest in the enterprise, serving on the Board of Directors continuously from its organization until his death. He was also a Director of the Northwestern National Bank and of the Minnesota Loan and Trust Company. He was President of the L. S. Gillette Company and the Plymouth Investment Company, both dealing in Minneapolis real estate, and of the Chippewa Land and Pasture Company.

During the last twenty years of his life, Mr. Gillette was engaged in and devoted much time to matters of public interest. He served as Chairman of the Building Committee of the United States Chamber of Commerce when its home at Washington, D. C., was constructed at a cost of $2 500 000. He was

* Memoir prepared by James B. Gilman, M. Am. Soc. C. E.

also an Honorary Vice-President of the Chamber and was a Delegate of that Society to International Conferences in Paris, France, and Rome, Italy.

His loyalty to his Alma Mater was evidenced by his action in joining with another alumnus in financing from their personal resources the employment of Mr. Cass Gilbert, of New York, to lay out a comprehensive plan for the enlargement of the campus. This plan has been maintained in the great development that has taken place in the last few years at the University of Minnesota. By his foresight and personal contribution, it is building toward a harmonious and well-designed arrangement instead of erecting a number of buildings with no definite comprehensive plan.

As an active member of Trinity Baptist Church, at Minneapolis, Mr. Gillette took great interest in its work, as well as in the work of other religious institutions in Minnesota. He was a Trustee of Carleton College, Northfield, Minn., and of Pillsbury Academy.

Mr. Gillette was one of the substantial business men of Minneapolis, prominent in all matters advocated for the betterment of the city. His later years were more devoted to business affairs than to engineering, but he was always keenly interested in engineering projects and in the Society. He was an extensive traveler and had a host of friends in all parts of the world. His death occurred at his winter home in Natchez, Miss., and he was buried in Lakewood Cemetery, Minneapolis.

Mr. Gillette was married to Louise E. Perkins, of Minneapolis, in 1878. He is survived by his wife, a son, George L. Gillette, and three daughters, Mrs. Marion Anson and Mrs. Harry C. Piper, of Minneapolis, and Mrs. J. Gilbert Burton, of Mobile, Ala.

He was a member of the Engineers' Club of New York, the University Club of Chicago, Ill., the University and Minnesota Clubs of St. Paul, Minn., and the Lafayette, Minneapolis, and Minikada Clubs of Minneapolis.

Mr. Gillette was elected a Member of the American Society of Civil Engineers on June 7, 1899.

GEORGE SEARS GREENE, Jr., M. Am. Soc. C. E.*

DIED DECEMBER 23, 1922.

George Sears Greene, Jr., was born in Lexington, Ky., on November 26, 1837. His father, George Sears Greene, Past-President, Am. Soc. C. E., was not only a prominent soldier in the Civil War, but also an eminent engineer, who practiced considerably in the City of New York. From him, George Sears Greene, Jr., inherited his engineering aptitude, as well as his patriotic and soldierly characteristics. He possessed the personal qualities of a gentleman, both in his professional work and in his daily life, and exemplified markedly the fine and courtly ways of the professional man who never forgot himself under any circumstances.

Mr. Greene once remarked that his New England family connections made it imperative that he should attend Harvard University for his education. After successfully completing the Freshman year, he withdrew in 1857 to take up his engineering training in practice with his father. From that time until the end of his life, he was continuously engaged in the actual conduct of engineering work of magnitude.

Mr. Greene acquired his engineering training during the infancy of the profession. His earlier engagements were on the Croton Aqueduct of New York City and in railroad work in Cuba, as well as in copper mines in the Lake Superior District. The shifting head for engineers' transits, now universally used, was invented by Mr. Greene while he was engaged, from 1872 to 1876, on an extremely accurate survey of what is now the Borough of the Bronx. This invention was made to overcome difficulties in work over rough ground requiring both speed and accuracy.

The particular work which gave Mr. Greene his eminence was in practically creating the water-front of Manhattan Island. From 1875 to 1897, he was Chief Engineer of the Department of Docks of New York City. At the beginning of his term of office, there was no systematic method of constructing the bulkhead wall. The small range of ordinary tides—about 5 ft.—made it practicable to provide effective accommodations for shipping without closed docks or any other complicated work. He therefore retained the pier system, piers projecting out into the river with slips between. Along the marginal streets, he constructed the permanent bulkhead wall.

In order to design this wall, Mr. Greene made a thorough study of the river bottom to determine the depth of the underlying rock, which he found to be at varying depths. Particularly in the North River certain parts required unyielding foundations at points where the depths of water and mud were great. A few bulkhead walls had been built, but they were found to be temporary, inadequate, and unsatisfactory. Mr. Greene made a thorough examination by the most effective means at his command, including the use of divers, and found the bulkhead walls in such condition that their early reconstruction was imperative. In order that the plans which he proposed might receive suitable criticism and that he himself might get the benefit of the

* Memoir compiled from information on file at the Headquarters of the Society.

best professional advice, he requested authority from the Dock Commission to secure "the united wisdom and experience of disinterested engineers, to examine the condition of that portion of the wall referred to in this report, and advise what remedy if any should be applied to it." This action in discharging his duty in the most effective manner by calling to his aid such professionally skilled and experienced men as the late Gen. John Newton, U. S. A., Hon. M. Am. Soc. C. E., Gen. Q. A. Gillmore, U. S. A., M. Am. Soc. C. E., and William E. Worthen, Past-President and Hon. M. Am. Soc. C. E., was characteristic of the man.

The formal judgments of these prominent consultants combined with the results of Mr. Greene's own experience developed into the plans for the bulkhead wall which, it may be said without exaggeration, placed him, in professional prominence, practically at the head of American engineers in charge of dock and pier construction. The efficiency and permanence of this work are conclusively evidenced by the fact that the bulkhead walls which he built in those early years are still performing their functions with a minimum of maintenance and repairs. The combination of pre-cast concrete blocks with both vertical and brace piles was a conception of great merit for his purpose, and the test of many years of service has justified his confidence in that design. Those plans, supported by many years of successful experience, have demonstrated the soundness of Mr. Greene's professional judgment and the scrupulous care which he gave to the administration of the constructive work of his office.

The more credit may be given for the successful issue of such operations in those early days for the reason that Portland cement was then a foreign product, the only domestic cements available for subaqueous operations being such natural cements as the Rosendale brands. Furthermore, even where Portland cement was imported, there had been only a limited experience in determining the best composition and methods for its use in concrete work under water.

Subaqueous concrete construction on such a scale as that of the Department of Docks of New York City gave well defined and effective stimulus to this type of construction. Indeed, it is difficult with the present practically universal use of domestic Portland cements to realize the disadvantages under which pioneers like Mr. Greene had to work. In spite of this handicap, he developed a plan of bulkhead construction which is still used successfully.

The retention of the position of Chief Engineer of the Department of Docks for twenty-two years under changing administrations and other vicissitudes of office would have been impossible for one less qualified professionally, or less resolute in discharging the duties of the office to the best of his ability. It is superfluous to state that he had his official trials. Nevertheless, he stood the shocks unshaken and in no case did the consultants called to examine or review his work ever give any material criticism. On the other hand, he was many times professionally complimented both by the officials of the Department of Docks and by prominent consultants.

Earlier City authorities had committed a great folly in filling in a large area on the North River between Perry and West 22d Streets, thus practically

destroying, for commercial purposes, that part known as the Gansevoort and Chelsea Sections. These sections now contain the great piers that provide docking facilities for the largest ships entering the Port of New York. Mr. Greene was probably the strongest engineering critic of this so-called improvement of the water-front of Manhattan Island. He urged the Dock Commission to excavate the area and construct piers, which work, after much opposition, was undertaken and subsequently completed.

In 1895, the Commissioners of the Department of Docks appointed a Board of Consulting Engineers* with the request that the members thereof "review the improvements of the water front, and the plans therefor" and "submit with their report, such suggestions as they may deem proper for the betterment of such plans, with the view of securing a larger usefulness of the water front of the City of New York." This Board made several reports, the first of which was dated February 6, 1896. In this first report, the Consulting Engineers made the following observation:

"As the bulkhead wall is a retaining wall, which has to resist the horizontal thrust of the earth-filling behind it, and as both that earth-filling and the wall itself are floated in mud, the problem of providing a permanent construction becomes a very unusual one. To build a retaining wall on solid foundation is a simple problem. To build a wall to carry vertical weight only, on a soft foundation, is more difficult. To float a wall in mud, when that wall must also take a horizontal thrust, is a problem which can only be solved by care and experience, no formulas or mathematical rules being available. The wall, as now built, is a satisfactory solution of this problem. Your Board believes it to be a unique construction, one which is worthy of the most careful study, and which deserves the strongest commendation."

This statement of unqualified commendation from members of Mr. Greene's own profession was in substance a simple statement of some of the facts connected with the administration of his duties and must have been all the more gratifying to one who had spent the best years of his life in developing a great public work for the City of New York.

On December 27, 1897, the Hon. Henry F. Dimock, Dock Commissioner, made an interesting address at the "placing of a stone to commemorate the work of dock improvement between Charles and West 23d Streets, North River", in which he stated:

"This City owes a peculiar debt of gratitude to one man whose merit is only equalled by his modesty. That man is Mr. George S. Greene, Jr., the accomplished Engineer in Chief of the Department. This plan is his child. He originated it; and he alone. By his forceful advocacy he has kept it alive during all these years and has won for it the approval of all the many Commissioners that have come and gone, and all intelligent men who have given him a hearing. This is indeed his day of triumph. He has occupied his present position for more than twenty-two years, under commissioners of every shade of politics; always giving service of the utmost professional value. A man of the highest character, of an integrity so known of all men that no breath of suspicion has ever reached him. He has stood there as an object lesson to prove that sometimes the highest professional and personal qualifications are recognized, even in official life. His retention for such a period has done infinite honor to the long line of commissioners under whom he has

* This Board of Consulting Engineers consisted of Gen. Thomas Lincoln Casey, U. S. A., the late George S. Morison, Past-President, Am. Soc. C. E., and William H. Burr, M. Am Soc. C. E.

served. It is certainly the hope of all who are interested in the water front that the time may be far distant when the City must be subjected to the loss of his services."

This commendation from a prominent member of the Department of Docks is a marked tribute to Mr. Greene's professional character and efficiency in the discharge of his duties.

Although Mr. Greene's connection with the Department of Docks did not extend beyond the year 1897, the effective plans which he developed for the bulkhead wall of the Harbor of New York have continued in use to the great advantage of the marine commerce seeking the port.

After his withdrawal in 1897 from the position of Engineer in Chief of the Department of Docks, Mr. Greene continued in the active practice of his profession as a Consulting Engineer. His success in the water-front constructions of New York City caused his services to be sought by those public interests requiring similar works.

Probably the most prominent of those services was performed as a member of the Advisory Board of Engineers,* appointed by the Governor of New York State, for the construction of the Barge Canal. The late John A. Bensel, Past-President, Am. Soc. C. E., State Engineer during the life of this Board, sought the constant co-operation of its members, in which Mr. Greene's long experience with large public work was of unusual value. It was during this period that the Scotia Dam, the Hinckley Dam, the heavy work at Medina, the dams on the Oswego River, and other large features of the Barge Canal work, were built.

Mr. Greene was an Honorary Member of the American Institute of Architects. His club memberships also included both the Harvard Club and the Century Association, and in the latter his fine personal qualities made him a perfect centurion. His genial companionship was a special attraction in that galaxy of interesting personalities of whom he was a popular associate for many years.

He was a genuine lover of Nature and a true sportsman, and thoroughly enjoyed fishing on a preserve in the mountains of Pennsylvania and shooting on the waters of Long Island Sound.

He was a life-long member of the Protestant Episcopal Church. His religious faith was true and unvarying and was marked by the same consistency and steadfastness that characterized his whole life.

Mr. Greene was elected a Member of the American Society of Civil Engineers on December 4, 1867, the year of its re-organization. He served as a Director from 1882 to 1884, and also in 1886; as Vice-President in 1885; and as Treasurer from 1888 to 1890. During all these years he served the Society disinterestedly and faithfully and maintained his interest in its development until his death. Not many years ago he called attention with special pride to the fact that three generations of his immediate family had served as members of its Board of Direction.

* This Advisory Board discharged its duties during nearly four years, 1911 to 1914, immediately prior to the completion of the work, and in addition to Mr. Greene, consisted of Messrs. Mortimer G. Barnes, Joseph Ripley, T. Kennard Thomson, and William H. Burr, Members, Am. Soc. C. E.

JOHN WILSON HAMILTON, M. Am. Soc. C. E.*

DIED APRIL 16, 1924.

John Wilson Hamilton was born in Wishaw, Scotland, on April 16, 1870, and came to this country with his father when he was thirteen years of age. He received his technical education at Cooper Institute, New York, N. Y., having been graduated in the Class of 1898, with the degree of Bachelor of Science, receiving at the same time the Cooper Medal. In May, 1890, he received the degree of Civil Engineer.

His first position which he held from July, 1886, to May, 1892, was with F. W. Davis and Brother, of Brooklyn, N. Y. He was engaged principally in general foundry work, including the furnishing and erection of iron work for buildings and, also, in 1891, on the construction of the Company's new plant.

From 1892 to 1893, Mr. Hamilton had charge of the Carrère and Haas Iron Works, in Brooklyn, and under his supervision the steel work for a number of buildings which included the Puck Building in New York, was manufactured. He was employed by the Empire Iron Works from 1893 until 1894, in charge of estimating and shop work. This Company was engaged in furnishing and erecting steel work for buildings.

In May, 1894, Mr. Hamilton became President of the Sherman Iron Works, of Brooklyn, which Company furnished and erected steel work for buildings, including a number of fireproof residences. From 1895 to 1896, he was employed by J. R. F. Keely and Company, General Contractors, on the construction of the German Herold Building in New York and also on work on the Croton Aqueduct.

In 1896, Mr. Hamilton accepted a position as Designer and Estimator in charge of work, with the Hay Foundry and Iron Works, but resigned in February, 1897, to become associated with Milliken Brothers, Incorporated. While with this firm, his work was divided into three periods: From 1897 to 1899, he had responsible charge of the detailing of all kinds of steel structures, from sugar-mills to office buildings; from 1899 to 1901, he made estimates for and designed bridges and buildings; and from 1901 until 1908, he held the position of Contracting Manager for the Company.

After the failure of Milliken Brothers, Mr. Hamilton organized the Hamilton and Chambers Company, of which he was President. The work of the firm consisted principally of engineering, manufacturing, and contracting. He was connected with this organization until his death on April 16, 1924, after an operation for goitre. He is survived by his wife, Mrs. Emma W. Hamilton, a son, Robert B. Hamilton, and a daughter, Elizabeth Hamilton.

Mr. Hamilton was a man of many lovable qualities which endeared him to every one with whom he came in contact, in business as well as socially. He was a Shriner and a member of many clubs, including the Engineers Club, New York, Whitehall Club, and several golf clubs.

Mr. Hamilton was elected a Member of the American Society of Civil Engineers on July 10, 1907.

* Memoir compiled from information furnished by Hamilton and Chambers Company and on file at Society Headquarters.

EDMUND HAYES, M. Am. Soc. C. E.*

DIED OCTOBER 19, 1923.

Edmund Hayes, the son of Gustavus Hayes and Sarah C. Shaw Hayes, was born in Farmington, Me., on May 15, 1849. He attended the preparatory schools of his native State, taught school for a time, and entered the Scientific Department of Dartmouth College where he remained for two years. He then entered the Civil Engineering Department of the Massachusetts Institute of Technology, from which he was graduated in the Class of 1873.

Shortly after leaving college, Mr. Hayes was employed with the old Erie Railroad, Western Division, on general engineering work and barometric observations with a view to reducing grades; later, he was employed by the Passaic Bridge Company of Passaic, N. J. In 1875, he became connected with the firm of Morrison and Field, Engineers and Bridge Contractors.

When the Central Bridge Company was organized in 1878, Mr. Hayes became a co-partner with George S. Field, M. Am. Soc. C. E. Later, this Company was merged with the Union Bridge Company. These companies had a most successful career and between 1878 and 1891 built many important bridges in all parts of the world, among them the Michigan Central Railroad Cantilever Bridge over the Niagara River at Niagara Falls, which, at the time, was one of the longest steel spans.

In 1891, the Union Bridge Company was absorbed by the United States Steel Corporation and now forms an integral part of the present American Bridge Company. On the sale of the Union Bridge Company, General Hayes became interested in the electrical development of water power at Niagara Falls and with other Buffalo capitalists built the third power plant on the Canadian side, that of the Ontario Power Company, which was sold later to the Canadian Government. With the same group of capitalists he organized the Niagara, Lockport and Ontario Power Company which distributes power from Buffalo, N. Y., throughout the western and central portions of the State.

In 1914, General Hayes retired from active business life, and previous to his death, he lived quietly in Buffalo, when not traveling in the United States, Europe, Egypt, and Palestine. He listed traveling and fishing as his chief recreations.

He was a member of the Buffalo Club, and of the Country Club, and a Director in the Society of Fine Arts and other institutions, such as the Marine Trust Company, and also a Trustee of the University of Buffalo.

General Hayes was active in the affairs of St. Paul's Protestant Episcopal Church, of Buffalo, having served as Senior Vestryman. He was well known for his works of charity, both public and private, and gave generously to local charities. He was also a contributor to Dartmouth College and the Massachusetts Institute of Technology.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

He acquired his honorary military title of General while serving as Chief of Engineers of the military forces of the State of New York on the Staff of Governor Alonzo B. Cornell. In 1913, Dartmouth College conferred on him the degree of Master of Science.

Edmund Hayes loomed large in the public life of Buffalo, although he had never held public office, nor sought it. He was one of that rare class of first citizens whose reputations filter through the community from those who know them intimately. In this manner, the people of Buffalo had gained a high opinion of General Hayes' ability, liberality, and public spirit. He had been engaged in large affairs all his life and through his own enterprise had done much to upbuild the city. He was particularly valued as a counselor and his advice was much sought. His death was a distinct loss to the community.

On April 30, 1878, he was married to Mary H. Warren, of Buffalo, by whom he was survived.

General Hayes was elected a Junior of the American Society of Civil Engineers on April 3, 1878, and a Member on March 5, 1884.

JOSEPH HOFMAN, M. Am. Soc. C. E.*

DIED APRIL 8, 1924.

Joseph Hofman was born in Detroit, Mich., on April 28, 1869. His parents moved to Toledo, Ohio, in 1870, where his father, Samuel Hofman, became a member of the firm of Black and Hofman, later known as The Alexander Black Cloak Company.

Joseph Hofman received his education in the public schools and High Schools of Toledo. His first engineering work was as a Rodman and Draftsman in the County Surveyor's Office in 1884. He continued in this capacity until November, 1886, when he joined the Engineering Corps of the Toledo, St. Louis and Kansas City Railroad ("Nickel Plate Road"). He assisted in the spectacular task of widening, in one day, the 237 miles of track between Toledo and Frankfort, Ind.

In 1890, Mr. Hofman returned to the County Surveyor's Office as Assistant, remaining until 1892. In 1893, when he was only 23 years of age, he was elected County Surveyor, and served until 1898.

From 1898 until 1900, Mr. Hofman was Assistant City Engineer of Toledo, in charge of street paving. During 1901 and 1902, he worked on various surveys, including those for the proposed water supply system for Toledo, and for the proposed cross-town railroad connecting the Wabash and the Wheeling and Lake Erie Railroads through Toledo. He served as Assistant City Engineer until 1905, when he was re-elected County Surveyor, serving until 1909.

Subsequently, Mr. Hofman was engaged, until his death, in private civil engineering work. He specialized in city and suburban surveys and developments, and platted many of the best known sub-divisions of the City of Toledo.

One of his many important accomplishments was that of bringing about the re-establishment of the Ohio and Michigan State Line; this was done in 1915, through his efforts personally and as Chairman of the Legislative Committee of the Toledo Society of Engineers.

Mr. Hofman was unmarried. He is survived by a brother, Mr. A. L. Hofman, and a sister, Mrs. Ella Freeland, both of Toledo.

He was a Past-President of the Toledo Chapter, American Association of Engineers; a member of the Toledo Rotary Club, Toledo Chamber of Commerce, Toledo Lodge No. 53, Benevolent and Protective Order of Elks, Knights of Pythias, B'nai Br'th, and Rubicon Lodge, Collingwood Chapter, and Grafton M. Acklin Council, A. F. and A. M.

Mr. Hofman was elected a Member of the American Society of Civil Engineers on August 9, 1920.

* Memoir prepared by George N. Schoonmaker, Assoc. M. Am. Soc. C. E.

ALBERTO ANGEL IBARGUEN y PI, M. Am. Soc. C. E.*

DIED JULY 9, 1924.

Alberto Angel Ibargüen y Pi was born in Matanzas, Cuba, on May 31, 1885. He attended the Provincial Institute of Matanzas where he completed his "secondary instruction" and from which he obtained the degree of Bachelor of Arts and Sciences. He became a Land Surveyor in 1902. He afterward entered the National University at Havana from which he was graduated as an Architect in 1908 and as a Civil Engineer in 1909. Mr. Ibargüen distinguished himself at the University, having been an honor student in the Department of Engineering.

From 1902 to 1909, Mr. Ibargüen was engaged in private professional work, first as Surveyor, and then as Architect and Engineer. In 1910, he entered the Department of Public Works as Auxiliary Engineer in charge of a highway from Guane to Mantua, in the Province of Pinar del Rio, Cuba. He was rapidly promoted to higher positions, and, in 1914, was "provisionally" appointed Chief Engineer of the District of Pinar del Rio.

Mr. Ibargüen held that position until 1923, when he was transferred to a position as "First Class Engineer" in the Central Office of the Department of Buildings of Public Works, in Havana, Cuba, which position he held at the time of his death. He is survived by Mrs. Ibargüen and four children.

Mr. Ibargüen was a thorough and conscientious worker. During his long term as Chief Engineer in the District of Pinar del Rio his administrative duties left him little time for purely technical matters; nevertheless, he contributed personally to many discussions, especially of reinforced concrete construction which he had adopted as his specialty.

He was of a kind disposition and made friends of all who met him. Prominent citizens of Pinar del Rio and members of the "Revista de la Sociedad Cubana de Ingenieros" have requested that a 15-m. highway arch bridge designed by him and yet to be built, be named "Puente Alberto Ibargüen", as a token of appreciation of his merits as an Engineer and a citizen.

He was a member of the Cuban Society of Engineers and for many years acted as its delegate in the Province in which he resided.

Mr. Ibargüen was elected an Associate Member of the American Society of Civil Engineers on June 23, 1916, and a Member on June 16, 1919.

* Memoir prepared by Francisco José Gaston, Assoc. M. Am. Soc. C. E.

ALBERT WILLIAM JOHNSTON, M. Am. Soc. C. E.*

DIED SEPTEMBER 8, 1924.

Albert William Johnston, the son of Thomas Hunter Johnston and Anne Metcalf Johnston, was born on what was formerly Old Fort Hill in Boston, Mass., on March 4, 1853.

Mr. Johnston received his education in the public schools of Boston, having been graduated from the English High School in 1869. He then entered the Massachusetts Institute of Technology from which he was graduated in 1873 as a Civil Engineer.

Immediately following his graduation, Mr. Johnston was employed on surveys for the Philadelphia and Delaware Railroad, near Philadelphia, Pa. During the winter of 1873 and the spring of 1874, he was engaged as Instructor in Mechanical Drawing in Boston. In the summer of 1874, he was connected with the American Dredging Company and the Pennsylvania Steel Company as Assistant Engineer, and the winter and spring of 1875 he again spent in Boston as Instructor in Mechanical Drawing.

In June, 1875, Mr. Johnston accepted a position as Clerk and Draftsman to the General Superintendent with the Pittsburgh, Cincinnati and St. Louis Railway Company, at Pittsburgh, Pa., continuing as such until 1878 when he gave up office work for field location. In 1879, he was made Assistant Engineer and was attached to the office of the Chief Engineer, in charge of the construction of masonry bridges.

During the winter of 1879 and 1880, Mr. Johnston was engaged in the resurvey of the Jeffersonville, Madison, and Indianapolis Railway, in Indiana, but in March, 1880, he went to Tombstone, Ariz., as Engineer and Superintendent of the Sycamore Spring Water Company, where he designed and constructed a reservoir and piped the water supply into this town of 8 000 inhabitants.

From January to April, 1882, he served as Chief Engineer in charge of maintenance for the Toledo, Delphos, and Burlington Railway Company, and from April, 1882, to June, 1883, as General Superintendent of Construction with the Leavenworth, Topeka and Southwestern Railway Company in Kansas.

From April, 1884, to January, 1889, Mr. Johnston was Division Engineer of Maintenance of the Eastern Division of the New York, Chicago and St. Louis Railway, with headquarters at Cleveland, Ohio. Five years later, he was appointed Division Superintendent and, subsequently, General Superintendent, General Manager, and, on July 15, 1916, Assistant to the President. He was retired in December, 1917, after thirty-four years of active service with the New York, Chicago and St. Louis Railway Company, but continued his residence in Cleveland until his death on September 8, 1924.

* Memoir prepared from information on file at the Headquarters of the Society.

In December, 1880, Mr. Johnston was married to Flora Kimball, at Steubenville, Ohio. Mrs. Johnston, with one daughter, Mrs. Huntington Smith, survives him.

He was actively interested in all railway affairs, having served as President of the American Railway Engineering and Maintenance-of-Way Association in 1908, and also of the American Railway Guild. On his retirement from active service in 1917, Mr. Johnston received more than three hundred letters from those with whom he had been associated during his professional career, one letter stating that his initials, "A. W. J.", stood for "Always with Justice".

Mr. Johnston was elected a Member of the American Society of Civil Engineers on March 6, 1895.

HOWARD MURFREE JONES, M. Am. Soc. C. E.*

DIED MARCH 31, 1924.

Howard Murfree Jones, son of William Rucker and Susan Catherine (Johnson) Jones, was born at Murfreesboro, Tenn., on October 9, 1874. He acquired his technical education at Vanderbilt University, Nashville, Tenn., and at Union College, Schenectady, N. Y., and was graduated from the latter institution in 1895 with the degree of Bachelor of Science in Civil Engineering.

Early in his college life, Mr. Jones' natural industry and thrift were exhibited by the devotion of much of his leisure to remunerative occupation, without neglecting his studies. For several years, he was both student of engineering and private secretary to the Dean of the Engineering Department. He thus defrayed a considerable part of the cost of his education, working apparently under a handicap, but gaining in reality a valuable training in resourcefulness and self-control, not commonly prescribed in the college curriculum. These sterling qualities, so early displayed, were retained in later years and contributed much to the high character, independence, and sense of responsibility, that dominated his whole professional life.

Immediately after graduation from Union College, Mr. Jones began his career as a Civil Engineer in the employ of the Nashville, Chattanooga & St. Louis Railway, in Nashville. He continued in this service for seven years, rising to the positions of Assistant Engineer and Chief Draftsman. In the summer of 1902, he became Assistant Bridge Engineer of the Louisville Bridge and Iron Company, in Louisville, Ky., where he remained about one year. During that time, he gave special attention to the design of bridges and steel structures. In 1903, he returned to Nashville as Bridge Engineer of the Nashville, Chattanooga & St. Louis Railway and filled that position until 1907.

From 1907 to 1913, Mr. Jones was engaged in private practice, as a Consulting Engineer, in Nashville. Although his practice included investigations and reports in other fields of engineering, he specialized in the design of steel and concrete structures, and his reputation as a Bridge Engineer, during this time, became well established. All his work was done with a thoroughness and ability that won for him the confidence of clients, contractors, and associates.

In 1908 and 1909, as Engineer for the Cumberland River Bridge Commission, Mr. Jones was charged with the design and erection of two important steel and concrete bridges over the Cumberland River, at Nashville. The structures were built at a cost of $1 000 000 and constitute an enduring monument to his skill in the design, and his ability in the direction, of engineering works of magnitude.

In 1912, Mr. Jones was employed by the Niagara Alkali Company, as Consulting Engineer, in the design and construction of a large chemical plant of reinforced concrete, at Niagara Falls, N. Y. The approbation which rewarded his labors in this work, on the part of both the owners and the contractors, was indicative of his success in the profession.

* Memoir prepared by John Y. Bayliss, M. Am. Soc. C. E.

In 1913, pursuant to an amendment of the Act to regulate commerce, known as the Valuation Act, the Interstate Commerce Commission appointed a Board of Engineers as the nucleus of an organization to make a valuation of all interstate common carriers in the United States. The Board was composed of five engineers, appointed from five districts into which the country was divided. Mr. Jones was the appointee from the Southeastern States and, with his four colleagues, entered actively into the work to which the last ten years of his professional life was dedicated. As a member of the Engineering Board, he brought to this task a knowledge of engineering practice, an acquaintance with railroad construction, and a capacity for comprehensive investigation, that admirably fitted him for the responsibility that devolved on this important body. He was a faithful and influential member at all board meetings held to formulate the methods and create the personnel by which the work was to be accomplished. Under his direction, the inventories and engineering reports for the Southern District were prepared, comprising about 50 000 miles of steam railroads and the corresponding lines and properties of the Western Union and Postal Telegraph Systems.

In 1921, when the field work in the five districts was finished, the Engineering Board and the district organizations were abolished. Mr. Jones was then made Supervising Engineer in charge of the completion of the engineering work for all districts, with headquarters in Washington, D. C., and had almost finished the task when his unexpected death occurred. On March 31, 1924, he was attending a hearing in the valuation of the New York, Philadelphia and Norfolk Railroad, in the offices of the Commission in Washington, when he was suddenly stricken and died within a few minutes. His death occurred in the prime of his life and at the zenith of his career as an engineer.

Howard Jones had a love for his profession and a zeal for his work that mark the successful man in all walks of life. He had a keen mind, a high purpose, and a loyalty to principle, which are distinctly attributes of the Engineering Profession; but he had also a judicial sense, not commonly possessed by engineers, which enabled him to penetrate some of the intricacies of the law in its application to engineering problems. He was a rapid reader, with a retentive memory, and a taste for literature and history, as well as scientific knowledge.

He was an exemplary citizen, a genial companion, and a good husband and father. He gave freely of his time and talents to progressive movements, whether professional, political, or social.

For many years, the Engineering Association of the South, of which he was a Past-President, owed its usefulness and activity in a large measure to his persistent efforts. At the time of his death, he was a member of the Cosmos Club, in Washington, and was President of the In-Com-Co Club of the Interstate Commerce Commission. He was also an active member of the Church of the Disciples.

On June 8, 1899, Mr. Jones was married, in Nashville, to Marion Woodall Tucker, of Kansas City, Mo., who, with four daughters, survives him.

Mr. Jones was elected a Member of the American Society of Civil Engineers on June 1, 1909.

WILLIAM VOORHEES JUDSON, M. Am. Soc. C. E.*

DIED MARCH 29, 1923.

William Voorhees Judson was born in Indianapolis, Ind., on February 16, 1865, and spent his boyhood in that State. His family was descended on the one side from earliest New England stock and on the other from the first Dutch settlers of New Amsterdam.

At the age of seventeen, he went to Harvard University, but while at home on a vacation during his second year he won a competitive examination for appointment to the United States Military Academy. Having been graduated third in his class in June, 1888, he was appointed a Second Lieutenant in the Corps of Engineers, U. S. Army, and served as an officer of that Corps, during the remainder of his life, reaching the grades of First Lieutenant in 1893, Captain in 1898, Major in 1906, Lieutenant-Colonel in 1912, and Colonel in 1917. During the World War he served as a Brigadier General.

On his appointment to the Corps of Engineers, Lieutenant Judson was assigned to duty with the Engineer Battalion and attended the Engineer School of Application, from which he was graduated in 1891. In the spring of that year, at her home in Lexington, Ky., he was married to Alice Carneal Clay; the marriage was characterized by unswerving mutual devotion until the day of his death.

For the next few years, Lieutenant Judson acted as Assistant Engineer to various officers in charge of river and harbor improvements on Lake Erie, on the Upper Mississippi, and at Galveston, Tex. This was followed by about 18 months as Adjutant of the only Battalion of Engineers the Army then boasted. In 1898, General Lawton requested that Captain Judson be appointed Adjutant General of the troops which Lawton commanded. The Chief of Engineers, however, refused to let him go on the ground that he could not be spared from strictly engineering duties. This decision was a bitter disappointment to Captain Judson, and he could never quite forgive or forget it.

In 1898, he was made Recorder of the Board of Engineers and from July, 1899, to August, 1900, he served as Chief Engineer and President of the Board of Public Works of the Island of Porto Rico under the Military Government which obtained there immediately after the war with Spain. This was the most important work which he had yet done, and the excellence of his accomplishment is recognized in the Annual Report to the War Department by the Governor of Porto Rico, Gen. George W. Davis, who said of Captain Judson: "He has shown marked ability as an organizer, administrator, and as an Engineer."

Following his year in Porto Rico, Captain Judson was for eighteen months in charge of river and harbor improvement and fortification construction on the Gulf Coast and in Georgia and Alabama; and for three years in Washington, D. C., as an Instructor in the Engineer School, a member of the River and Harbor Board, and Assistant to the Chief of Engineers.

* Memoir prepared by William P. Wooten, M. Am. Soc. C E., and Clay Judson, Esq., Chicago, Ill.

In March, 1904, Captain Judson was sent as Military Observer with the Russian Army in the Russo-Japanese War. At the battle of Mukden, he was captured with a large body of Russian troops by the rapidly advancing Japanese and returned to America through Japan. His official report on the war was filled with his distinctive independence of thought and sound common sense. Some of the conclusions which he drew were not generally concurred in at that time, but their soundness was thoroughly demonstrated by the lessons of the World War.

After his return from Russia, Captain Judson was placed in charge of the River and Harbor District at Milwaukee, Wis., where he had an opportunity to put into use the concrete caissons which he had invented several years before. This device, patented and donated to the Government, was described by him in a paper* read before the Western Society of Engineers. He regarded his invention of these caissons as the most valuable single piece of construction work that he ever did for the Government. It was partially in recognition of this service that Harvard University conferred on him in 1911 the honorary degree of Master of Arts.

In March, 1909, Major Judson was appointed Engineer Commissioner of the District of Columbia. For four years he filled that office with distinction. During that period he succeeded in effecting substantial economies and improvements in the lighting service, the water distribution, the street cleaning, and other engineering activities. He designed and constructed on a large tract of land in Virginia a model work-house for 600 District prisoners. He earnestly and constantly urged the passage of legislation leading to better regulation of corporations, public utilities, and private building operations, to a termination of abuses in the insurance business, to the adoption of a definite program for the execution of public works, the acquisition of parks, and the extinction of the public debt, and to the inauguration of many other measures of improvement and reform. Much of the legislation which he advocated was enacted into law during his term of office. The wisdom of many more of his suggestions has been recognized by the passage of appropriate laws during more recent years.

On the completion of his tour as Engineer Commissioner of the District of Columbia, Colonel Judson was ordered, at the special request of Gen. George W. Goethals, U. S. A. (now Major-General U. S. A. (*Retired*)), M. Am Soc. C. E., to Panama as Assistant Division Engineer of the Atlantic Division, which included the work at Gatun. Colonel Judson remained in Panama only one year, when he was relieved at his own request.

He spent the next three years in charge of the River and Harbor District and Division at Chicago, Ill., and, in 1916, was ordered to Baltimore, Md., to take charge of the river and harbor and fortification work in that District. He had been there nearly a year, when war was declared against Germany.

Colonel Judson was immediately ordered to raise, organize, and command a regiment of Engineers to be taken to France as soon as possible, but before

* "Concrete-Steel Caissons: Their Development and Use for Breakwaters, Piers and Revetments," *Journal*, Western Soc. of Engrs., Vol. XIV (1909), p. 533.

he could begin to execute this order, it was countermanded, and he was attached to the Root Commission which was then about to start to Russia. When the Root Commission returned to America, Colonel Judson was left in Russia as Military Attaché and Chief of the American Military Mission. At about the same time he received his commission as Brigadier General in the National Army. Shortly before this he had been decorated by General Brussiloff with the order of St. Anne, 2d Class, with Swords, and a little later received from the Kerensky Government the Order of St. Stanislaus, 1st Class, with Swords.

During his stay in Russia, General Judson saw what the diplomats failed to see, that the revolutionary spirit in the Army, the lessening of the authority of the officers, and the crumbling of discipline were certain to bring chaos in their wake. As early as August 7, 1917, he cabled the War College: "At least an even chance that Russia will go out of the war within a few months. Our larger war plans should be made accordingly." He was entirely unable to convince the American and Allied diplomatic representatives of the danger, however, and they constantly reported, to December, 1917, that Russia would stay in the war to the bitter end.

In spite of the discouragement which met him at every turn, and the lack of understanding of the Ambassador, General Judson, without any official support, contributed most valuable advice and assistance to the unofficial anti-Bolshevik crusade which possibly delayed by several weeks the advent of this faction to the control of the Government. After the accession of the Bolsheviks to power, General Judson urged in vain that the inevitable be recognized and that every effort be made to guide the exercise of that power along lines most in the interest of America and the Allies. The best he was able to accomplish, however, was to obtain from the Ambassador a grudging consent to arrange for an interview with Trotsky on the eve of that official's departure to negotiate an armistice with the Germans. Trotsky was apparently moved by General Judson's carefully prepared argument and finally said that he would demand a long armistice, the detention of enemy troops along the front, and no exchange of prisoners or products. Although he was unable to obtain these favorable terms, he did resist for months the imposition of the harsh German peace and thus delayed the utilization of the full German power on the Western Front until after the United States had become prepared to put forces into the field to help meet it. How much this delay contributed to the final victory of the Allies may be left for the careful historian to estimate.

General Judson returned to the United States in January, 1918. After a full appreciation of his work as Chief of the Military Mission in Russia, the President awarded him the Distinguished Service Medal for his services there.

During the spring and summer of 1918, he commanded the 38th Division at Hattiesburg, Miss., and was officially commended for the progress made in the training and discipline of the Division while under his command.

In the late summer of 1918, he was assigned to the command of the Port of Embarkation of New York. For the next two months he was responsible for the embarkation and transportation of 500 000 men and for the shipment of

the greater part of the supplies for the Expeditionary Forces, and, in addition, was constantly responsible for the well-being of from 100 000 to 200 000 troops. The period was at the very height of the influenza epidemic which made troop movement so difficult and harrowing a task.

A slight attack of influenza and the strain under which he had been constantly working led General Judson to suspect that a physical breakdown was near. Until the day the Armistice was signed, however, he worked at high pitch, giving all his energy to the task before him. Immediately the war was over, he felt compelled to consult a physician, and it was found that his heart was greatly enlarged and that any long continuance of his life was improbable. Nevertheless, after four months in bed, although realizing that his end was always near, he resumed his task and accomplished much work of value.

Back in Chicago, a Colonel of Engineers again, in charge of river and harbor improvement, he further developed his study of that city's particular problem and worked out a plan for future harbor development in the Wolf Lake region, which he called "Illiana Harbor." The plan was outlined in some detail in a paper read before the Western Society of Engineers on November 7, 1921.*

In December, 1921, while on a trip down the Mississippi River, there came a failing of his heart more pronounced than any he had yet suffered. He fought a hard and patient battle against his fatal trouble, but was never able to recover sufficiently to resume his work, and in August, 1922, he was retired from active service. A final attack the following spring was not to be withstood, and he died on March 29, 1923, at Winter Park, Fla.

The qualities which most distinguished William Voorhees Judson were perhaps the fearless independence of his thought, the soundness of his judgment, and a never failing sense of humor. This combination of characteristics made him the best of companions, for every event took on a freshness and new meaning when seen through his eyes. Naturally, such qualities endeared him to his many friends in all parts of the world, although they made him powerful enemies as well. Yet his independence was never either radical or careless. He had a boundless regard for history and science, and the value of his work and ideas was due to the fact that in every case they rested on the firmest foundations. He was careful to an extreme and did not believe in taking unnecessary chances; however, his conservatism, because combined with a remarkable creative faculty, led to considerable accomplishment—accomplishment which would have been greater still had he not been stricken at the very time in life when ripened experience had added so much to his productive capacity.

General Judson was elected a Member of the American Society of Civil Engineers on November 4, 1896.

* "Illiana Harbor," *Journal*, Western Soc. of Engrs., Vol. XXXVII (1922), p. 201.

FRANK MONTGOMERY KERR, M. Am. Soc. C. E.*

DIED MAY 5, 1924.

Frank Montgomery Kerr, the son of Richard Hulton Kerr, of Nassau, New Providence, Bahama Islands, and Priscilla McCarty Kerr, of Cork, Ireland, was born in New Orleans, La., on February 28, 1851. His grandfather, Louis Kerr, the first of the family to come to the America's, was Governor General of the Bahama Islands.

Frank Montgomery Kerr was reared in New Orleans, and educated first by private tutors, and then in the public schools of that city. He entered the Louisiana State University and Agricultural and Mechanical College in 1867, from which he was graduated in 1871, with the degree of Civil Engineer.

Following his graduation, he returned to New Orleans, and entered the employ of the State, under Gen. Jeff M. Thompson, Chief State Engineer of Louisiana, with the title of Assistant to Engineers. Later, he served as Private Secretary to the Chief Engineers of the Louisiana Levee Company, the late Caleb G. Forshey, M. Am. Soc. C. E., Mr. G. W. R. Bailey, Col. T. S. Hardy, and Mr. John V. Van Pelt. In 1877, under the administration of Governor Francis T. Nicholls, the duties of the Louisiana Levee Company were absorbed by the newly created Board of State Engineers, and Mr. Kerr was selected as Chief Draftsman for the Board, under the late B. M. Harrod, Past-President, Am. Soc. C. E., Chief State Engineer. He served in this capacity until 1882, under the late Benjamin H. Greene and Henry B. Richardson, Members, Am. Soc. C. E., after which he was promoted to the position of Secretary of the Board of State Engineers. In 1888, he was promoted to be Assistant State Engineer and Member of the Board of State Engineers. He served in that capacity, under Major Richardson, until April, 1904, when upon the appointment of Major Richardson to the Mississippi River Commission, Mr. Kerr was selected to fill the unexpired term of the Chief State Engineer of Louisiana. He was then successively re-appointed Chief State Engineer of Louisiana in 1904, 1908, 1912, 1916, and 1920. He held this position at the time of his death in 1924, and would have been re-appointed to fill his sixth term had he lived. Although he never took an active part in politics, Mr. Kerr held the position of Chief State Engineer of Louisiana for 20 years, and for 53 years continuously served the same Department of the State.

In 1878, Mr. Kerr was married to Almaide Kate Bonnabel, of New Orleans. He is survived by Mrs. Kerr and four children: Mrs. William Andrew Jackman, Frank McCarty Kerr, Charles Macdonald Kerr, Assoc. M. Am. Soc. C. E., of New Orleans, and Mrs. Bartlett U. Sims, of Bryan, Tex. He died while visiting Mrs. Sims, at her Texas home.

For a number of years he was identified with the State National Guard of Louisiana, serving as a Lieutenant in Battery B of the Louisiana Field Artillery. He afterward served for several years as Inspector of Rifle Practice,

* Memoir prepared by Charles M. Kerr, Assoc. M. Am. Soc. C. E.

with the rank of Major, on the staff of Gen. John Glynn, and also on the staff of Governor Heard, with the rank of Colonel. Major Kerr was an expert marksman, and engaged, as member of his regimental team, in National rifle matches, in which he gained individual distinction.

He was a Founder and Past-President of the Louisiana Engineering Society, and always took an active part in its deliberations and management. He was also a member of several fraternal organizations.

Major Kerr was possessed of a kindly disposition and engaging personality, and his friends were numbered among the prominent men, not only of his city and State, but of the professional world at large, in which he was directly interested. He was considered an authority on matters pertaining to the control of the flood waters of the Mississippi River and its tributaries, and he devoted his life and energies to the problems of holding in check the destructive forces of the mighty "Father of Waters". He gained distinction for the success of his efforts, and with the courage of his convictions, held steadfast to his opinions and ideals, and passed out of this life with the confidence, love, and respect of his fellow men.

Major Kerr was elected a Member of the American Society of Civil Engineers on October 2, 1901.

FRANK BARR KNIGHT, M. Am. Soc. C. E.*

DIED OCTOBER 12, 1924.

Frank Barr Knight was born in Worcester, Mass., on February 13, 1872. He was graduated from Worcester Polytechnic Institute in 1892, in the Department of Civil Engineering. A few weeks prior to his graduation he was engaged to enter the employ of the Lidgerwood Manufacturing Company which Company was his sole employer during his professional career, from 1892 until his death. He first entered the Cableway Department as a Draftsman and, later, was trained as a Cableway Erector. Mr. Knight's work brought him in contact with every phase of cableway design, construction, and operation. He made several improvements, notably in excavating buckets and aerial dumping appliances.

From 1894 to 1896, he was in charge of field construction on the Chicago Drainage Canal where many Lidgerwood cableways were used. In 1908, Mr. Knight was made Manager of the Chicago Branch of the Lidgerwood Manufacturing Company, in which position his talents for business management and engineering design were conspicuous factors in the success of his undertakings. He was the author of several articles on the subject of cableways, notably one entitled "Mining Lime Rock." †

Mr. Knight was President of the National Drainage Congress which met in Cairo, Ill., in January, 1916, having specialized in the study of drainage and reclamation projects, as well as equipment for construction and excavation.

He was a Thirty-second Degree Mason and member of Kismet Temple, Brooklyn, N. Y. He was a member of the Union League Club and the Engineers Club of Chicago, the Western Society of Engineers, and the Machinery Club of New York, N. Y.

He died suddenly of heart failure on October 12, 1924, at his residence at Highland Park, Ill. For a great many months Mr. Knight had been in poor health, although he was able to attend to his business duties to the last.

He is survived by his wife, an only son, Frank Burrows Knight, and the host of friends whom he had attracted by his genial and generous nature.

Mr. Knight was elected an Associate Member of the American Society of Civil Engineers on September 4, 1901, and a Member on September 3, 1912.

* Memoir prepared by Spencer Miller, M. Am. Soc. C. E.

† *Mines and Minerals*, August, 1899.

JAMES ALBRO LAWRIE, M. Am. Soc. C. E.*

DIED MAY 21, 1924.

James Albro Lawrie was born in Hamilton, Ont., Canada, on September 23, 1862, the son of Thomas A. and Lucinda (Atkins) Lawrie. He came to the United States early in life and was educated at Detroit, Mich.

Mr. Lawrie began active work as an engineer in supervising the construction of docks and grain elevators on Lake Erie ports under his father. From 1886 to 1888, he was engaged in the installation of power plants in various places in the Eastern States.

In 1893, he went to Sault Ste. Marie, Mich., as City Engineer, which position he held until 1902. He designed the sewerage and water systems and supervised the various improvements that were carried out in that rapidly growing city. He worked at that time with the late Rudolph Hering, M. Am. Soc. C. E., and with Charles F. Loweth, Past-President, Am. Soc. C. E., as Consulting Engineers.

Resigning this office in 1902, Mr. Lawrie accepted a position in Louisville, Ky., in charge of foundry and machine shop work and the erection of power plants in Ohio and Kentucky. Subsequent to 1905, he worked in Duluth, Minn., first as Engineer for the National Iron Company, which position he held until May, 1906, and thereafter with the Duluth, Missabe, and Northern Railway Company as Assistant Engineer in charge of the office work under the Chief Engineer. This position he filled with great credit until his death on May 21, 1924. During his service with this Company his natural engineering ability showed itself in the design of some of the largest iron ore docks on the Great Lakes, and many other important structures.

In 1884, Mr. Lawrie was married to Mary Boyd, at Mercer, Pa.; he is survived by his wife and one son, Wallace R. Lawrie.

During his life in Duluth, Mr. Lawrie was an active participant in everything that tends to the upbuilding of a good community. He took special interest in outdoor life and was a pioneer in advocating the conservation of natural resources. He was the leader in establishing the Local Chapter of the Boy Scouts of America and gave freely of his time and energy to this work, being a member of the Regional Committee No. 10, comprising the States of Minnesota and Dakota. He was a member of the Izaak Walton League of America and was one of the founders of the Superior National Forest Association, in the work of which he took a very active part. He was also a member of the Duluth Engineers' Club.

Mr. Lawrie was elected a Member of the American Society of Civil Engineers on March 12, 1918.

* Memoir prepared by W. H. Hoyt, M. Am. Soc. C. E.

NELSON PETER LEWIS, M. Am. Soc. C. E.*

DIED MARCH 30, 1924.

Nelson Peter Lewis, the son of John Neher and Christina Jane (Nelson) Lewis, was born at Red Hook, Dutchess County, N. Y., on February 1, 1856. He received the degree of Bachelor of Arts from St. Stephen's College, Annandale, N. Y., in 1875, and was graduated as a Civil Engineer from Rensselaer Polytechnic Institute in 1879. The honorary degree of Doctor of Laws was conferred on him by St. Stephen's College in 1911.

Mr. Lewis achieved an enviable reputation both in the United States and abroad as a municipal engineer and city planning expert, and as such his services were invaluable to the City of New York. He began his engineering practice, however, in railroad location, first with the Denver and South Park and the Denver and Rio Grande Railroads in Colorado, in 1881, and from 1882 to 1884, with the Vicksburg, Shreveport and Pacific Railroad in Louisiana, following which, for three months, he was Supervisor of Track with the Delaware and Hudson Railroad.

On July 1, 1884, Mr. Lewis joined the Engineering Staff of the Department of City Works of Brooklyn, N. Y., as Assistant Engineer on an extension of the Water-Works System. From this time, with the exception of the period from July 1, 1886, to July, 1889, when he was with the Central Railroad of Georgia, in charge of railroad location and construction and of the planning of terminal and water-front improvements in Alabama and Georgia, he remained in continuous municipal service, first with the City of Brooklyn until the formation of Greater New York in 1898 and after that date with the Consolidated City.

In 1894, the late Alfred T. White, Commissioner of City Works of Brooklyn, appointed him Chief of the Bureau of Local Improvements, which step Mr. Lewis has described, as follows:

"Alfred T. White was Commissioner of City Works of Brooklyn in 1894 and 1895. One of the Bureaus of the Department had to do with street improvements. Its head was a man whose previous occupation had been—I do not remember what—plumber or possibly an undertaker or some quite honest business, but it did not conspicuously fit him to administer a Bureau of the City Government which had to do exclusively with engineering work. Mr. White believed that an office of this kind should be administered by an engineer and he designated me as its head, with the idea that it should be placed upon an engineering basis; but he did more than this—he gave me a free hand in the re-organization of the office and in the selection of a staff of capable assistants."

In this position Mr. Lewis was able to build up an engineering staff that contained many men who have since been active in the various engineering organizations of the city.

* Memoir prepared by the following Committee: Arthur S. Tuttle, *Chairman*, Richard S. Buck, and J. Waldo Smith, Members, Am. Soc. C. E., and George H. Pegram and George S. Webster, Past-Presidents, Am. Soc. C. E.

For the four years beginning with 1898, he was Chief of the Brooklyn Bureau of Highways; it was during this period and under his leadership that the substitution of the present type of smooth pavement for the old system of cobblestone highways in that Borough, was undertaken.

Mayor Seth Low noticed the progress which was being accomplished by Mr. Lewis in Brooklyn and appointed him as the first Chief Engineer of the Board of Estimate and Apportionment of New York City in March, 1902. He held this office through successive administrations until his retirement in December, 1920. This position called for reports on all engineering projects brought before the Board of Estimate and Apportionment for approval, and demanded a combination of professional skill, judgment, integrity, and tact, with which Mr. Lewis was exceptionally well equipped. His activities were largely responsible for the orderly development of the plans under which the growth of the city was shaped, as well as for the successful functioning of the various agencies created for carrying on the vast engineering work. Undoubtedly, his judgment often saved the city vast sums of money.

Mr. Lewis served on a number of committees or commissions appointed by the Mayor or the Board of Estimate and Apportionment, such as the New York City Improvement Commission, the Commission to Investigate the Subject of Municipal Lighting Plants, the Heights of Buildings Commission, and the Committees to Investigate and Report on Methods of Snow Removal and Garbage Disposal. He represented the City of New York at the International Road Congresses held in Paris, France, in 1908, in Brussels, Belgium, in 1910, and in London, England, in 1913, and at the International Congress of Cities in Ghent, Belgium, in 1913.

In 1910, he visited many European cities and reported on the character and control of their sub-surface structures and the use of pipe galleries for this purposes.

Mr. Lewis was one of a delegation of nine American engineers representing the National Engineering Societies, who visited France in December, 1918, at the invitation of the Société des Ingénieurs Civils de France and several of the Cabinet Ministers, to confer with committees of a French Congress under the chairmanship of M. Millerand, on problems of reconstruction following the World War.

After his retirement from the city service in 1920, Mr. Lewis had several months of well-earned rest in California and then became, in May, 1921, Director of the Physical Survey of the Regional Plan of New York and Its Environs under the auspices of the Sage Foundation. He began studies of physical and other conditions bearing on metropolitan development within an area of 5 528 sq. miles centering around New York City, and continued in this work (later, as Director of the Engineering Division) until his death.

He was the author of "The Planning of the Modern City", the second edition of which was published in 1923, and had contributed freely to the engineering press on matters relating to city planning, highways, and general civic improvement.

Nelson P. Lewis was one of the few engineers who have attained, in the grueling field of public service, both marked eminence in his profession and the admiration and affection of all those associated with him. He completed nearly two-score years of work in this service, filled with many unusual changes and difficulties, still carrying an unblemished reputation for integrity, loyalty, and professional efficiency. He had vision, appreciation, and interest in a high degree. He was genial, kindly, and helpful to all who sought his counsel or aid, an outstanding engineer, a warm friend and a delightful companion.

Mr. Lewis is survived by his wife, Mrs. Minnie McLean Lewis, and one son, Harold M. Lewis, M. Am. Soc. C. E., who is Executive Engineer for the Regional Plan of New York and Its Environs.

Mr. Lewis was the first President of the Municipal Engineers of the City of New York and Past-President of the American Society for Municipal Improvements, of the American Road Builders' Association, of the Brooklyn Engineers' Club, and of the Brooklyn Municipal Club. In 1919-20, he was President of both the National Conference on City Planning and the American City Planning Institute. He was a Trustee of the Brooklyn Polytechnic Institute, a Trustee and Past Secretary of the Adirondack League Club, and a member of the Engineers' Club of New York. He was also a very active member of the Flatbush Dutch Reformed Church and served, at two different times, as an Elder on its Consistory.

Mr. Lewis was elected a Member of the American Society of Civil Engineers on October 2, 1895. He served as Director from 1904 to 1906 and as Vice-President in 1918 and 1919.

HORACE LOOMIS, M. Am. Soc. C. E.*

DIED MARCH 8, 1925.

Horace Loomis was born in Binghamton, N. Y., on July 14, 1840. He received his early education in private schools and Binghamton Academy. In 1860, he was graduated from the State Normal School at Albany, N. Y. (now known as Teachers' College), and served for two years as a Tutor in Mathematics in the Albany Academy. He was graduated in 1865 from Rensselaer Polytechnic Institute, Troy, N. Y., with the degree of Civil Engineer.

For the next ten years Mr. Loomis practiced his profession, devoting most of that time to railroad work. During this period, he served as an Assistant Engineer with the Lehigh Valley Railroad Company, the Lehigh Coal and Navigation Company, the Erie Railroad Company, and several other railroad companies.

He went to Mount Vernon, N. Y., in 1869 as Chief Engineer under the Commission appointed to improve the old White Plains Road between Mount Vernon and White Plains, N. Y., and several contributory roads. In this position he supervised the extension and rebuilding of many streets and highways.

In 1875, Mr. Loomis entered the employ of the City of New York as Assistant Engineer in the Department of Water Supply, and among other things made surveys for the New Aqueduct from Croton Lake to New York. He also made a survey of the Housatonic River for an additional water supply and did similar work in connection with the Bronx River.

In 1880, he became Chief Engineer of Sewers of New York City and, with the exception of a brief tenure of the office of Commissioner of Street Cleaning in 1890, served in that capacity until 1914, when he was placed on the retired list, incidentally being the first Municipal Engineer to receive a pension.

Mr. Loomis found time notwithstanding his engrossing professional engagements to participate actively in civic and business affairs. He was President of the old Village of West Mount Vernon for two years, just previous to its consolidation with Mount Vernon. He served for two years as Trustee of the Village of Mount Vernon, as Town Auditor, and was elected Excise Commissioner, but did not serve his term. He was President for eleven years of the Peoples Bank of Mount Vernon, which was later incorporated as the First National Bank of Mount Vernon. He also served for more than a year as President of the Mount Vernon National Bank.

He was a member of the Municipal Engineers of New York, the Engineers' Club of New York, the Alumni Association of the State College of Teachers, the Alumni Association of Rensselaer Polytechnic Institute, and the Men's Club of Trinity Protestant Episcopal Church of Mount Vernon.

It seems almost unnecessary, in view of the above record, to call attention to the fact that notwithstanding his noteworthy professional achievements and technical ability, he had those rare broad and sterling human qualities which

* Memoir prepared by Charles Warren Hunt, Secretary Emeritus, Am. Soc. C. E.

made him a factor in the political, business and social activities of the city in which he lived for more than half a century.

Mr. Loomis is survived by two sons, Horace Benjamin Loomis, of New York, and Romeo F. C. Loomis, of Mountainberg, Ark.; and four daughters, Miss Harriet J. Loomis, of Mount Vernon; Mrs. Anne V. Stubbings, of Bernardsville, N. J.; Mrs. Mary Alice Sellars, of Kaycee, Wyo.; and Mrs. Emma L. Bishop, of Garden City, Long Island; and by two sisters, Miss Eleanor Loomis, of Binghamton, N. Y., and Mrs. George W. Robinson, of New York.

The following resolutions in regard to the death of Mr. Loomis were adopted by the Board of Direction of the Society at its meeting in Cincinnati, Ohio, on April 20, 1925:

"*Whereas,* The Board of Direction of the American Society of Civil Engineers has learned with regret of the death of Horace Loomis; and

"*Whereas,* Mr. Loomis has been a member of this Society since November 5, 1879, and has served with distinction as Director from 1910 to 1912, and as Vice-President during 1912;

"*Therefore Be It Resolved:* That it hereby expresses its deep sorrow at his passing, its appreciation of his great work and exemplary character, and its sympathy with his family in their loss; and

"*Resolved Further:* That these resolutions be spread on the Minutes, and a copy forwarded to the members of his family."

Mr. Loomis was elected a Member of the American Society of Civil Engineers on November 5, 1879, and served as a Director of the Society from 1910 to 1912 and as Vice-President in 1912.

CHARLES OSCAR McCOMB, M. Am. Soc. C. E.*

DIED JULY 27, 1924.

Charles Oscar McComb, the son of Oscar and Isabel Tuck McComb, was born in New York, N. Y., on October 9, 1859. His education was acquired in that city and Brooklyn, N. Y., chiefly in private schools.

His first professional engagement was with the Pennsylvania Railroad Company, and from 1880 to 1884, he was employed in the Engineering Department of the Erie Railroad Company. In 1884, he engaged in private practice in Bloomfield, N. J., serving as Engineer for that village in 1885 and 1886. During part of 1886 and 1887, he was Assistant Engineer with the Carthage and Adirondack Railroad Company, to which, after a brief service with the Lehigh and Schuylkill Haven Railroad Company, he returned as Principal Assistant Engineer.

In 1889, Mr. McComb became Engineer and Superintendent of the Magnetic Iron Ore Company, which position he left to become associated with the firm of Hinds and Bond, Engineers and Contractors, in charge of the construction and operation of water-works at Chatham, Ont., Canada, until 1892. In that year, he resumed his private practice in Carthage, N. Y. He designed and built water-works and sewerage systems for a number of cities and villages in Northern New York, water-power developments on the Black, Oswegatchie, and Grass Rivers, and relocated part of the Gouverneur and Edwards Railroad. In 1894, he became City Engineer of Watertown, N. Y., and, in 1898, also Engineer of the Water Board of that city. From that time until 1904, considerable paving, sewer construction, and large bridge work was done under his direction.

Mr. McComb relinquished this position to become Division Engineer of the State of New York, with headquarters at Syracuse. During his occupancy of this office, he had charge of both canal and highway work. Foreseeing the great expansion in highway construction, he entered this field of contracting on his own account in 1907, and was immediately successful. This occupation was continued until his death, except for the interruption of the World War period, when road building was suspended. During this time, he was engaged as Chief Engineer for the contractor on the construction of the Housing Project at Watertown, N. Y., under the Housing Bureau of the United States Department of Labor.

Mr. McComb had an agreeable and pleasant personality which gained him many friends, his scrupulously honorable and conscientious character enabling him to retain them through life. He was a loyal member of the Protestant Episcopal Church and, in politics, of the Republican Party.

In 1890, Mr. McComb was married to Caroline E. Benton, of Adams, N. Y., who survives him.

Mr. McComb was elected a Member of the American Society of Civil Engineers on May 2, 1900.

* Memoir prepared by J. H. Weidman, Assoc. M. Am. Soc. C. E.

HIRAM ALLEN MILLER, M. Am. Soc. C. E.*

DIED NOVEMBER 2, 1923.

Engineer, economist, and Christian gentleman, Hiram Allen Miller represented the highest ideals of professional engineering practice. After his graduation from Sheffield Scientific School at Yale University, he steadily rose by his unobtrusive yet keen initiative through the various steps of experience to a prominent place as consulting engineer in hydraulic and water-works construction. During his engineering activities, he had charge of the design and construction of public works the value of which was about $25 000 000.

Hiram Allen Miller was born at Williston, Vt., on June 3, 1853. He was the son of Charles Eliot and Emily (Clark) Miller. His ancestor, John Miller, who, with his father, Daniel, came from England not long after the arrival of the *Mayflower,* was killed by the Indians at Springfield, Mass., in 1657. On his mother's side, also, the line of descent in Massachusetts goes back to early Colonial days.

Working on the farm in summer and attending the district school in winter, and after that a country academy, Mr. Miller laid the foundation for his persevering and industrious career. He entered Yale University with the Class of 1875, but left at the end of the first term because of the death of his father. He returned the following year in the Class of 1876, and entered Sheffield Scientific School, taking the course in Civil Engineering. He received prizes for excellence in mathematics, physics, and other studies in his Freshman year, and was Vice-President of his Class. He took a divided prize in mathematics in his Junior year. Later, he was President of his Class and also of the Christian Union. After his graduation in 1876 with the degree of Bachelor of Philosophy, he returned for a year for post-graduate work and also served as Instructor in Mathematics. In 1896, he was awarded the degree of Civil Engineer by the Sheffield Scientific School.

Like so many able men in the Civil Engineering Profession, Mr. Miller's earlier years present a panorama of frequent changes from the completion of one job to the commencement of another. Leaving Yale, he took up railroad engineering and from 1878 to 1884 was located in Southern Iowa, Nevada, Texas, and Louisiana. In 1880, following two years of railroad construction in the West, he became Chief Draftsman of the American Railway Improvement Company, then building the New Orleans Pacific Railway, and rose quickly to the position of Bridge Engineer and then Division Engineer in charge of 150 miles of construction. Later, he served as Division Engineer of the New Orleans and Mississippi Valley Railroad, including location and the difficult construction of the levee around the Bonnet Carre Crevasse. In 1884, he was Division Engineer of the New Orleans Pacific Division of the Missouri Pacific Railroad. His railroad experience involved responsible charge of about 300 miles of steam railroads and some electric lines.

* Memoir prepared by Sanford E. Thompson, M. Am. Soc. C. E.

From 1884 to 1892, Mr. Miller was located in Des Moines, Iowa. During these earlier years, also, he made excursions for brief periods into teaching and life insurance. In 1889, he was Chief Engineer on the location and construction of the Des Moines Belt Line Railway. In 1892, he accepted a position as Assistant Engineer on the construction of the Main Channel of the Chicago Drainage Canal, in charge of seven miles of construction. Three years later, in October, 1895, he came to Massachusetts and became Engineer in charge of the Reservoir Department of the Metropolitan Water-Works then under construction, directing the work of construction of the Wachusett Reservoir which has a capacity of 65 000 000 000 gal., at Clinton, Boylston, and West Boylston. He designed and built, subject to the approval of the Chief Engineer, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., the North Dike of this reservoir, an earth dam of unique design nearly 100 ft. high in its maximum section. The work also involved forestry work from the planting of the seed in the seed beds to the final planting in the field, the improvement of the old forests, and the construction and maintenance of roads.

Then came the period of service from 1903 to 1910, as Chief Engineer of the Charles River Basin Commission, in charge of the construction of the Charles River Basin. This work required not only engineering supervision in construction, but careful hydraulic studies, and covered the design and construction of two locks, large sluices for flood flow, a draw-bridge, 85 ft. wide, with two street-car tracks, two large marginal conduits of the nature of intercepting sewers, a complicated electrical distribution plant, a steam distribution plant, and other features. The larger of the two locks was 45 ft. wide and 450 ft. long in the clear, with 18 ft. in clear depth below mean low tide.

This improvement transformed the Back Bay District of Boston, Mass., and the Cambridge water-front from areas contiguous to an ill-smelling tidal basin—extending with its muddy banks to Watertown and Newton—to beautiful residential and park sections facing a pleasing reservoir of constant level, suitable for boating and other water sports.

After the completion of the Charles River Dam in 1910, Mr. Miller opened an office for private practice as Consulting Engineer in Boston, and from that time until his death, he served various private projects and many municipalities, including Dalton, Lenox, Needham, Pittsfield, and Weymouth, in Massachusetts; New Britain and Bristol, in Connecticut; and Rutland and Vergennes, in Vermont. He also was called on for expert testimony in a number of damage suits and for the taking of land and water by various cities. His work at Pittsfield, Mass., included a masonry dam for water supply, about 100 ft. high, the cost of which totaled about $800 000.

As a contribution to the World War, he served as Resident Engineer, District No. 1 (New England), for the Emergency Fleet of the United States Shipping Board.

To recount his engineering services is to give only the professional side of his life. By his integrity, his quiet keenness, and his lovable character, Mr. Miller made many life-long friends and won a place of respect and honor in the community. In his church at Newton Highlands, Mass., he was complet-

ing at the time of his death twelve years' service as Deacon. He was a lifelong Democrat, not because of parentage, but because, as a student of economics, he believed in the fundamental principles of free trade. As an economist of the highest rank, the late President Wilson also appealed to his ideals.

Two years before his death, Mr. Miller had had a major intestinal operation. He had never fully recovered his health, and his death on November 2, 1923, from nervous collapse was the indirect result of this operation. He was buried in Williston Cemetery, at Williston, Vt.

Mr. Miller was married on October 7, 1884, to Martha A. Buckingham, of New Milford, Conn., who died on July 4, 1914. His second wife, to whom he was married on December 26, 1917, was Bertha Marion Converse, of Newton Highlands, the daughter of Martha and Conrad R. Converse. Of his first marriage, three children survive. The daughter, Anna Irene Miller, is Assistant Professor of English Literature at Goucher College, Baltimore, Md. One of the sons, Buckingham Miller was graduated, like his father, from the Sheffield Scientific School of Yale University in 1910 and is an engineer in New York City. Hiram Allen Miller, Jr., a graduate of the Classical School of Yale University, 1917, is also an engineer engaged in industrial management for a large clothing plant in Chicago, Ill. Both sons served as commissioned officers in the World War.

Mr. Miller was a member of the Boston Society of Civil Engineers, Western Society of Engineers, American Water Works Association, New England Water Works Association, American Forestry Association, Massachusetts Highway Association, New England Historical Genealogical Society, American Economics Association, American Academy of Political and Social Science, Boston Chamber of Commerce, Congregational Club, St. Botolph Club, and Yale Club of Boston. He was a contributor to the *Journal* of the New England Water Works Association, the *Engineering News-Record,* and other technical periodicals.

Mr. Miller was elected a Member of the American Society of Civil Engineers on May 6, 1896.

PAUL COOK NUGENT, M. Am. Soc. C. E.*

DIED JULY 15, 1924.

Paul Cook Nugent, the son of Perry and Amanda Nugent, was born in New Orleans, La., on January 4, 1871.

He received from Roanoke College, Salem, Va., the degrees of Bachelor of Arts in June, 1889, and Master of Arts in 1894. In the fall of 1889, he entered Rensselaer Polytechnic Institute, at Troy, N. Y., being awarded the degree of Civil Engineer in 1892.

For several years following his graduation from Rensselaer, Mr. Nugent was connected with the Engineering Department of the Southern Pacific Lines, at Houston, Tex. In 1896, he accepted a position as Instructor of Civil Engineering in Lafayette College, Easton, Pa., which position he filled for one year, going from there to Syracuse University, Syracuse, N. Y., as head of the newly created Department of Civil Engineering.

He built up his Department at Syracuse, remaining as its head and taking an active part in the academic life of the University until January, 1919. During this time, Professor Nugent was also engaged in consulting practice. He was engineer in charge of the construction of the Syracuse University Stadium which cost $500 000; he also planned and supervised the construction of one of the most completely equipped hydraulic laboratories in the United States.

In January, 1919, Professor Nugent moved to Arizona on account of ill health and, for four years prior to his death, served as Professor of Civil Engineering in the University of Arizona. On his death, the Faculty of that Institution, by resolution, paid him the following tribute:

"In Professor Nugent we all recognized a chivalrous, cultivated, and noble minded gentleman; a scholar, a scientist, a friend and neighbor, gentle, generous, and considerate; an able and sympathetic teacher, tempering as he did, a lofty idealism with good sense and practical judgment. His patience and courage in bearing up under hopeless suffering and affliction has been an inspiring example of heroism to all his associates on the faculty."

The Faculty of the College of Applied Science of Syracuse University, also passed the following resolutions:

"The Faculty of the College of Applied Science, Syracuse University, has learned with sincere regret of the passing of our former associate, Professor Paul Cook Nugent.

"Professor Nugent was Professor of Civil Engineering and Head of the Department in this institution from 1896 to 1919, when he resigned on account of ill health.

"Professor Nugent was always much beloved by his fellow associates and by the students and his leaving Syracuse was a source of regret to all that knew him.

"His work as a teacher and author was always noteworthy. His devotion to all of his college work won him the highest regard of all."

* Memoir prepared by J. C. McClure, M. Am. Soc. C. E.

On September 12, 1899, he was married to Mary Louise Logan, of Salem, Va., who, with two sons, survives him.

In 1901, Professor Nugent edited a textbook on "Plane Surveying", for college use. He was a member of the following fraternities: Phi Gamma Delta, Tau Beta Pi, Phi Kappa Phi, and Sigma Xi.

Professor Nugent was elected a Member of the American Society of Civil Engineers on May 3, 1910.

LOCKE McINDOE PERKINS, M. Am. Soc. C. E.*

DIED JULY 2, 1924.

Locke McIndoe Perkins, the son of Marsh Olin and Clara McIndoe Perkins, was born at Windsor, Vt., on November 20, 1879. He entered Dartmouth College in 1897, from which he received the degree of Bachelor of Science in 1901, and the degree of Civil Engineer (Thayer School) in 1903.

Beginning in April, 1903, Mr. Perkins served the Northern Pacific Railway Company as Rodman and as Instrumentman on the construction of new terminals in Seattle, Wash., for two years. In April, 1905, he was promoted to the position of Resident Engineer on the construction of the Sykeston Branch Extension, and a few months later was again promoted and transferred to Duluth, Minn., as Assistant Engineer in Charge of Maintenance of the Lake Superior Division. In November, 1906, he returned to the Construction Department as Assistant Engineer on second-track line and grade revision between Garrison and Missoula, Mont., and the relocation of the White Pine Hill Line. In April, 1907, he was promoted to the position of Division Engineer, in charge of maintenance of the Northern Pacific, Lines East of Mandan, N. Dak. In February, 1910, he was made Engineer, Maintenance of Way, of all Lines East of Paradise, Mont., in general charge of maintenance and construction work, and in June, 1911, he was transferred to Tacoma, Wash., as Engineer, Maintenance of Way, of all Lines West of Paradise.

During this period, Mr. Perkins had general charge of the construction of new terminals and yards at Parkwater, Pasco, Auburn, Centralia, and Seattle (Hanford Street), Wash., as well as of a number of yard extensions, depots, etc., in addition to general supervision of all maintenance work.

In August, 1918, Mr. Perkins was made Corporate Engineer, in charge of the interests of the Northern Pacific Railway Company in matters of maintenance and construction during the period of Federal Control. In this capacity, he assisted in establishing a number of important fundamental principles used as a basis for the general settlements, between the Government and the carriers, of the claims arising from Federal Control and the Guaranty Period.

In March, 1920, he was made Assistant to the Chief Engineer, and in January, 1921, was promoted to the position of Executive Assistant to the President, continuing in this office until his death.

Mr. Perkins possessed to an unusual degree the keen, analytical mind necessary for the successful engineer. His fair and considerate dealing with his subordinates and with the many departments of the Railway endeared him to all his associates.

In private life, his kindly disposition and tactfulness made him a host of friends and a leader in movements for civic betterment. He was particularly

* Memoir prepared by H. E. Stevens, M. Am. Soc. C. E.

interested in helping the working boy improve his position; he assisted in organizing and acted as one of the first leaders of the Northern Pacific Employed Boys Club.

On February 11, 1908, Mr. Perkins was married to Ruth Roberts, of Devils Lake, N. Dak., who, with three sons, Locke, Donald, and Marsh, and one daughter, Ruth, survives him.

He was a member of the St. Paul Athletic Club, the Town and Country Club, the Benevolent Protective Order of Elks and of the Masonic Fraternity.

Mr. Perkins was elected a Member of the American Society of Civil Engineers on September 9, 1919, and was prominent in the activities of the Northwestern Section.

JULIUS PITZMAN, M. Am. Soc. C. E.*

DIED AUGUST 31, 1923.

Julius Pitzman, one of the most notable figures in the civic development of the City of St. Louis, died on August 31, 1923. His monument, more enduring than marble or granite, is the greater happiness, the larger industrial opportunity, of the city of his adoption.

He was born in Halberstadt, Prussia, June 11, 1837, the youngest of nine children. His early education was obtained at the Real Gymnasium in his native town.

Two of Mr. Pitzman's sisters, whose husbands had been strong sympathizers with the German Revolution of 1848, had been compelled to emigrate to the United States. Upon the death of his father in 1852, his mother decided to re-unite her family here. In preparation for this move, Mr. Pitzman devoted his entire time until May, 1854, to private study of English, French, and accounting.

The family arrived in New York, N. Y., after a voyage of six weeks and proceeded at once to the home of one of Mr. Pitzman's sisters near Milwaukee, Wis. Mr. Pitzman found employment as a clerk in the Post Office shortly after his arrival, but, on account of lack of opportunity for advancement, resigned the position in the fall of 1854 and moved to St. Louis, Mo., where he secured a position in a dry goods store at a salary of $8 per month. He later became connected with an agricultural implement company with an advanced compensation of $16 per month.

Commercial pursuits were not to his liking, and Mr. Pitzman entered the employ of his brother-in-law, Mr. Charles E. Salomon, who was Assistant to Mr. Henry Kayser, then City Engineer of St. Louis. He applied himself with characteristic energy to the study of mathematics and surveying under the guidance of Mr. Salomon and Mr. Kayser, and, in 1856, was appointed Deputy County Surveyor of St. Louis County which, at that time, included the City of St. Louis. In 1859, he opened an office in St. Louis for the practice of Surveying. His conspicuous ability in his chosen work became speedily apparent, and he rapidly built up a lucrative business.

Mr. Pitzman's Civil War record was one of notable activity in the service of his adopted country. He volunteered in 1861 and was appointed First Lieutenant of Engineers by General Fremont, in which capacity he built Fort No. 5, located immediately west of Lafayette Park in St. Louis. Upon the completion of this work, he was transferred to the command of Col. George Thom, Chief of Topographical Engineers, on the staff of General Halleck, and under his direction, made a topographic military map of St. Louis during the winter of 1861-62.

In April, 1862, a few days after the Battle of Shiloh, he was transferred to Pittsburg Landing, Tenn., and immediately proceeded with the construction

* Memoir prepared by Baxter L. Brown, W. E. Rolfe, and F. G. Jonah, Members, Am. Soc. C. E.

of a topographic map of the battlefield, on the completion of which he took part in the operations around Corinth, Miss.

In July, 1862, Mr. Pitzman reported to General Sherman, then commanding the Fifth Division of the Army of the Tennessee, and during the fall of that year made the necessary surveys and military maps for the defense of Memphis, Tenn.

He entered the command of Capt. J. H. Wilson, Chief of Engineers on the staff of General Grant, in October, 1862. As the Federal Government did not, at that time, grant commissions to engineer officers, General Sherman obtained from Governor Gamble of Missouri his appointment, on November 29, 1862, as First Lieutenant, Company D, Sixth Missouri Infantry. He was then detailed for duty as Engineer and Aide de Camp and took part, until seriously wounded on May 23, 1863, in the activities preceding the Battle of Vicksburg. He was promoted on April 5, 1863, to the rank of Captain, Company D, Sixth Missouri Infantry, on the staff of the Fifteenth Army Corps.

He did not recover from the effects of his wounds until the fall of 1863, when, on account of his physical condition, he was forced to resign from active service. Shortly thereafter, he was elected County Surveyor of St. Louis County.

In September, 1864, when General Price with a large force of Confederates threatened St. Louis, Mr. Pitzman again volunteered and was appointed Major and Chief Engineer of the First Division of Missouri State Militia and took an active part in the campaign against General Price. He was mustered out of service with the Militia on November 3, 1864, and resumed his office as County Surveyor, which he held until the separation of the City of St. Louis from St. Louis County, in 1876. At this time, he was appointed a City Surveyor and held the appointment until his death on August 31, 1923.

Mr. Pitzman's vision of the future of St. Louis began to show itself with the coming of peace. To enumerate all the public work in which he was actively interested, would mean virtually to write the history of the civic development of St. Louis since the Civil War.

He developed the privately controlled residence "place" so characteristic of St. Louis, and, in the face of ultra-conservative preference for straight lines and right angles in city streets, planned and promoted many of the finest residence districts in the city, with winding roadways and beautiful vistas. Vandeventer Place, then the most exclusive residence street in St. Louis, was laid out in 1870 on the site of Fort No. 6, built during the Civil War.

About the same time, he was active in securing the passage of ordinances providing for the widening of Jefferson and Grand Avenues, with the idea of establishing broad thoroughfares through and around the city. Neither project was carried out, because of the refusal of the City Comptroller to recommend appropriations. The city is paying to-day, and paying well, for its lack of appreciation of Mr. Pitzman's farsightedness.

Between 1872 and 1874, Mr. Pitzman represented a Citizens' Committee which advocated the lowering and bridging of the Pacific Railroad in Mill

Creek Valley, which has since become the city's greatest railroad center. Differences of opinion resulted in the reference of the problem to a Board of Engineers, and this Board not only reported favorably on the Pitzman plan, but elaborated it considerably.

O'Fallon, Carondelet, and Forest Parks, large tracts of waste land far beyond the existing city limits, were secured to posterity largely through Mr. Pitzman's foresight and initiative. He led the movement which resulted in the passage of an Act of the State Legislature authorizing their acquisition. City bonds were issued for the improvement of Forest Park, and, in 1874 and 1875, this incomparable recreation ground was planned and laid out under Mr. Pitzman's supervision as Chief Engineer, in conjunction with Mr. Kern, the Landscape Architect.

He was a member of the Board of Freeholders elected in 1875, to frame a new charter for the City of St. Louis, involving an extension of the city limits. He submitted to the Board a plan contemplating the inclusion of the whole of St. Louis County. This plan was rejected by the Freeholders, and Mr. Pitzman played a large part in the final determination of the new limits. One of his main contentions was that the River des Peres, for a large portion of its course, should be included within the city, as it was inevitable that the discharge of sewage into the stream would cause a nuisance, the abatement of which would devolve on the city. The wisdom of this course has since become apparent in connection with the recently adopted plan for the improvement of the river.

While acting on the Board of Freeholders, Mr. Pitzman was largely responsible for the establishment of the Board of Public Improvements, the administrative body of the city. The Charter provided that a majority of this Board should be engineers.

He early realized that the improvement of river-front property on both sides of the Mississippi River could not be adequately completed without the establishment of definite harbor lines. In 1903, he submitted a comprehensive plan to this end, which, with slight variation, was approved by the Secretary of War. Under this plan, the Cities of St. Louis and East St. Louis acquired possession of land valued at several million dollars.

Mr. Pitzman was mainly instrumental, during 1903 and 1904, in bringing about the establishment of the East Side Levee & Sanitary District, formed for the purpose of protecting industrial property in East St. Louis and adjacent towns in Illinois, from overflow of the Mississippi River, and to insure uninterrupted railroad communication with the East.

He was appointed, in 1901, with Mr. George Kessler, as engineer for the Louisiana Purchase Exposition, but withdrew because of differences of opinion on technical matters.

In 1917, Mr. Pitzman served as a member of a Board of Engineers appointed by the Commercial Club and Business Men's League, to make a study of industrial and commercial conditions in St. Louis. The report submitted by this Board recommended the expenditure of $62 000 000, most of which has since

been approved by the citizens of St. Louis through the authorization of bond issues.

Thus, in fragmentary outline, Julius Pitzman's life work is written. From his earliest youth to advanced age, he was continuously active in business and in contact with public affairs. He was that rare combination, a dreamer and a practical man. It was a fine tribute to a man of conspicuous ability when a prominent man of affairs in St. Louis said: "I know of no one who enjoys the confidence of his clients in higher degree than does Julius Pitzman."

On October 1, 1867, Mr. Pitzman was married, in St. Louis, to Emma R. Tittman, and to them were born a daughter and two sons. Mrs. Pitzman died on October 17, 1872. On March 31, 1879, he was married to Caroline Marsh Wislizenus, and to them were born four sons and one daughter.

He was an Honorary Member of the American Institute of Architects, and a member of the Engineers' Club of St. Louis, the Missouri Historical Society, the Academy of Science, the Loyal Legion of the United States, the Noonday Club, and the St. Louis Country Club.

Mr. Pitzman was elected a Member of the American Society of Civil Engineers on December 4, 1907.

ARCHIBALD OLIN POWELL, M. Am. Soc. C. E.*

DIED NOVEMBER 18, 1923.

Archibald Olin Powell, the son of Pardon Archibald and Sarah Woodruff Powell, was born in Milwaukee, Wis., on August 31, 1859. He received his early education in the public schools of Milwaukee, and this was followed by a collegiate course in Engineering at the University of Wisconsin, from which he was graduated in 1882 with the degree of Bachelor in Civil Engineering. In 1890, he received the degree of Civil Engineer from the same institution.

Commencing as early as the summer of 1875, and continuing through the succeeding summers of his college vacations, Mr. Powell served as Recorder on U. S. Lake and Mississippi River Surveys and as U. S. Sub-Overseer on Wisconsin River improvement work. Resuming this latter work on his graduation, he was soon given charge of the operation of the canal connecting the Fox and Wisconsin Rivers and of the Upper Lock on the Fox River. From 1882 to 1906, he was connected with the U. S. Engineer Office at St. Paul, Minn.—most of the time as Principal Assistant to the Engineer Officer in charge of that District. During this period, he was concerned chiefly with hydraulic investigations and improvement operations on the Chippewa, St. Croix, Minnesota, and Mississippi Rivers, and had charge of the design and construction of many important works.

In 1906, Mr. Powell came to Seattle, Wash., to prepare plans for and to report on the Lake Washington Canal Project, designed to connect Lake Washington and Lake Union with Shilshole Bay on Puget Sound by a lock and about four miles of ship canal. This investigation was made for a private corporation and, although the consummation of the project was delayed a few years, it was ultimately carried through, under direction of the U. S. Engineer Office, in large measure following the plans outlined by Mr. Powell. On the completion of this engagement, Mr. Powell established a permanent office in Seattle for practice as Consulting Engineer, specializing in hydraulics and more particularly in river and harbor work. This practice he continued until his entry into the military service during the World War. He was among the first of the civilian engineers to enter the Service, and among the last to leave, his connection with the military establishment extending from April, 1917, to July, 1920. After his discharge from the Army, he returned to Seattle to resume the practice of his profession in association with Joseph Jacobs, M. Am. Soc. C. E., under the firm name of Powell and Jacobs. This association continued until Colonel Powell's death, which occurred suddenly and unexpectedly on November 18, 1923, as result of a cerebral hemorrhage.

He was twice married. On February 6, 1882, he was married to Annit Hortense Maloy at Portage, Wis. His second marriage which took place in 1909, at Seattle, was with Mrs. Margaret Foster Powell, the widow of his elder brother, Brig.-Gen. Charles A. Powell, U. S. Army. Colonel Powell had only two children of his own, two sons by his first wife, but it was his pleasure also to help rear and educate the six children of his second wife, all of whom

* Memoir prepared by Joseph Jacobs, M. Am. Soc. C. E.

received collegiate training. He is survived by these eight children, five sons and three daughters, Mrs. Powell having died a month after Colonel Powell's death.

Colonel Powell always evinced a keen interest and actively participated in civic affairs, serving frequently on important committees of worthy civic organizations, and contributing valuable reports to them. He was ardently patriotic and much interested in military affairs. During the Spanish-American War, he organized one of the Companies, and was commissioned Captain, in the Second Regiment, U. S. Volunteer Engineers. During the World War, he was almost continuously attached to the office of the Chief of Engineers, U. S. Army, at Washington, D. C. He retired from the service with the rank of Lieutenant Colonel of Engineers.

Colonel Powell was a man of firm and positive character and opinions, and he always expressed these opinions with fairness and fearlessness. He was the soul of honor, a man of high ideals, an able engineer, and to those who enjoyed his intimate acquaintance, a most companionable and lovable character.

Among his fraternal and society affiliations were the following: American Legion, Association for the Advancement of Science, Canadian Institute of Engineers, Seattle Chamber of Commerce, Seattle Municipal League, Rainier Club, and Beta Theta Pi. He served one year as President of the Pacific Northwest Society of Engineers and one year as President of the Seattle Section of the Society.

Colonel Powell was elected a Member of the American Society of Civil Engineers on March 2, 1898.

GEORGE HENRY PRESTON, M. Am. Soc. C. E.*

DIED MARCH 17, 1924.

George Henry Preston, the son of G. H. Preston and Julia D. Preston, was born in Dansville, N. Y., on June 8, 1879. He was denied the advantages of a college education, but this loss was largely made up by close application in various positions which he held as a newspaper reporter, in business, and in engineering work. He attended the High School at Wilkes-Barre, Pa., for about two years, pursuing at the same time a night course in the Young Men's Christian Association. His work there attracted the attention of Mr. J. H. W. Hawkins, a local architect, who employed him as an Apprentice and Draftsman for about a year.

Mr. Preston's inclination led him to seek work of an engineering nature, and he obtained a position as Rodman in the Engineering Corps of the Lehigh and Wilkes-Barre Coal Company, which he held during 1898.

From 1899 to 1901, Mr. Preston was with the Electric Testing Department of the Hazard Manufacturing Company, Wilkes-Barre, in charge of repairs, insulated wires, and cables. The following year he worked as Draftsman and Checker with the Phœnix Iron Company, at Phœnixville, Pa., and, in 1903, was with the Baltimore Bridge Company as Draftsman in charge of structural detailing. For two years, 1903-1905, he was employed by the Cambria Steel Company at Johnstown, Pa., in the Structural Department, in charge of detailing steel work.

At this juncture, it became his lot to assume control of the Seward Coal Company and the Seward Brick Company of Johnstown, as President and General Manager, and for four years he carried on this business with much skill in the doubly difficult capacity of administrator and official. After this work was completed in 1909, he returned for a year with the Cambria Steel Company as a Designer.

Mr. Preston then turned his attention to the field of concrete engineering, working on general building construction with the Turner Construction Company for about eleven years, from 1909 to 1921, progressing through the various grades in the Engineering Department to that of Assistant Engineer. During the early part of this period, he took a special course in concrete engineering at Columbia University.

For a short time in 1911 he undertook a special type of engineering work in New York, N. Y., for the Otis Elevator Company, in charge of the design of air cushions and special foundations for the Woolworth Building and with N. M. Andrews, Architect, in charge of the design of the foundations for the Equitable Building.

In 1922, Mr. Preston was in business as Consulting Engineer, and in a few months had successfully established a working clientele, but receiving an offer from John Ferguson Company, Builders, of Paterson, N. J., he accepted the position of Engineer, which he held until his death.

* Memoir prepared by H. C. Paddock, M. Am. Soc. C. E.

Mr. Preston was an Engineer by nature and by practice. He had an exceptionally ingenious mind and was clever in solving difficult engineering problems. As a man, he was much loved by his associates, and is remembered as one who was always willing to impart his knowledge to those anxious to learn.

One of his characteristics was his love of a joke, but his pleasure was not the less when the joke was on himself. Of an even disposition and always courteous, he held the esteem of all who knew him. Although of a jovial nature, he was conscientious almost to a fault, and it was this characteristic, no doubt, that contributed largely to his death. Being in a run-down physical condition, he did not have sufficient vitality to withstand an attack of pneumonia, to which he succumbed after a week's illness.

He is survived by his wife, who was Lucy Brown, of Johnstown, Pa., together with a daughter and a son.

Mr. Preston was elected an Associate Member of the American Society of Civil Engineers on April 4, 1911, and a Member on November 21, 1921.

JAMES GEORGE ROSS, M. Am. Soc. C. E.*

DIED JULY 20, 1922.

James George Ross was born at Bethlehem, Pa., on January 30, 1878. He was graduated from Lehigh University in the Class of 1900.

For several years after his graduation, Mr. Ross was engaged in mining work in Pennsylvania. His first work for the Government was on the construction of the Plaquemine Locks connecting the Mississippi River with the Bayou Plaquemine, at Plaquemine, La. After supervising the construction of gun emplacements and an ammunition magazine at Fort St. Philip, La., he was placed in charge of the construction of the jetties in Southwest Pass, at the mouth of the Mississippi River, one of the most important and extensive works of its kind undertaken by the Government.

In 1908, Mr. Ross accepted the position of Assistant Superintendent of Dredging Operations under the Mississippi River Commission and was placed in charge of floating plant and the repair yard for the fleet of large hydraulic dredges used in the maintenance of low-water channels in the Mississippi River from Cairo to the head of the Passes below New Orleans, La.

In 1913, he left the Government Service to accept the management of the sand and gravel plant at Memphis, Tenn., of the Missouri Portland Cement Company. He remained in charge of this work until 1917, when he entered the First Officers' Training Camp at Fort Oglethorpe, Ga. On the completion of his training, he was commissioned a Captain and assigned to the 306th Engineers, 81st Division, participating in all the battles in which that Division took part.

On his return to Memphis in 1919, Captain Ross opened an office as Consulting Engineer and, later, became Managing Partner of Hilliard and Ross, Incorporated, Engineering Contractors.

His most notable characteristics were his force of character in handling men and his thoroughness in mastering the details of work. These characteristics are illustrated by the following incident in connection with his work on the Mississippi Jetties. On one occasion, the contractor suspended work on account of rough weather, stating that the waves were running too high to attempt to tow the timber mattress, which had just been completed, into position and sink it. Captain Ross told the contractor that if he would let him have the tug and crew, he would tow the mattress into position and sink it, which he did, thus demonstrating to the contractor that the work could be carried on under the then existing weather conditions and speeding up the progress very materially.

While in the full vigor of an active career Captain Ross met an untimely death on July 20, 1922, by stepping into the path of a moving train.

Captain Ross was elected a Member of the American Society of Civil Engineers on May 28, 1912.

* Memoir prepared by F. H. Hilliard, Esq., Memphis, Tenn.

WILLIAM HORATIO SANDERS, M. Am. Soc. C. E.*

DIED FEBRUARY 10, 1924.

William Horatio Sanders was born in Linlithgow, Scotland, on June 12, 1839. He was the youngest and only son of nine children. His mother died in Edinburgh, when he was still a little child, his father having moved to that city soon after his birth.

About 1850, Mr. Sanders, the elder, with his son and two daughters, came to America in a sailing vessel, being more than a month on the water. He opened a leather store in New York, N. Y., where his son's education was completed.

William Horatio Sanders entered the Army of the United States on August 7, 1862, and was honorably discharged on February 25, 1866, with the rank of First Lieutenant and Adjutant. He commenced the study of engineering about this time, and was employed in various engineering work until 1875.

From 1875 until 1887, he designed and constructed the dam and canal for a flour mill on the Pomme de Terre River, Minnesota. He reconstructed the Meeker (masonry) Dam, on the Chippewa River, Minnesota, and did much other work in connection with drainage and power plants in that State.

In 1887, Mr. Sanders moved to Southern California and devoted himself exclusively to hydraulic engineering. He designed and constructed the Mountain View Tunnel and the concrete pipe line from the tunnel at the mouth of San Antonio Canyon to the Mountain View Tract, Upland, Calif. On this line, he constructed a concrete siphon, which was said to be the first in California. He also designed and built the Fleming and Becket Tunnel and concrete pipe conduits, now known as the Indian Hill Tunnel and pipe line, near Claremont, Calif.

Mr. Sanders reconstructed the dam for the South Riverside and Temescal Water Companies, and was Engineer in Charge of Design and Construction for the well system at Lordsburg, Calif., and the concrete line from these wells to Covina, Calif., a distance of eight miles. He designed and built the sewerage system for the City of Pomona, Calif., as well as the outfall sewer and septic tank, which was the first application of the septic process in California.

He also designed and constructed the following concrete or masonry reservoirs: Currier, Richards, Loop and Meserve, Packard Vineyard Tract, Evergreen Tract, and the concrete-covered reservoir for the water supply for the City of Pomona, together with the concrete pipe lines for these reservoirs, all of which are in Los Angeles County.

The "Historical San Antonio Power Plant", the first with long-distance transmission, having a power line extending to Pomona and San Bernardino, Calif., was designed and constructed by him, as well as the Ontario Power Plant. There are nine tunnels on this supply pipe line. He also built the transmission line to Upland and Ontario, Calif.

For thirteen years, Mr. Sanders served as Chief Engineer for the Pomona Land and Water Company of Pomona, one of the largest water companies of Southern California, for which he had charge of all design and construction.

* Memoir prepared by J. H. Quinton, M. Am. Soc. C. E.

He was one of the Consulting Engineers for the United States Reclamation Service from May 1, 1903, until about 1918, and was connected with the design and construction of many of the large works of the Service, as well as those of the United States Indian Irrigation Service.

In the course of his engineering practice, Mr. Sanders designed and built many smaller domestic, irrigation, water and sewerage systems, in various cities and towns of Southern California.

He was married twice, his first wife having been Lorinda J. Fields. They had four children, who are still living: Mrs. Minnie Cronkhite, of Wapato, Wash., Mrs. Isabel Dibble, of Cannon Falls, Minn., William Francis Sanders, of Visalia, Calif., and Charles Horatio Sanders, of Los Angeles, Calif. Mrs. Sanders died on December 3, 1875. On September 19, 1878, he was married to Emma Adelia Ellsworth, an accomplished lady with literary tastes, who took an interest in all his work, and was a real help to him in much of it. She survives him. Their married life of more than 45 years was ideal, and constitutes a good example for the rising generation.

Mr. Sanders was a Mason for more than sixty years and was a great admirer of that Order. He took nearly all the degrees, was a Scottish Rite, Knights Templar, and Shriner of Al Malaikah Temple of Los Angeles. He was also a Member of the Military Order of the Loyal Legion of the United States, and a charter member of the Benevolent Protective Order of Elks, of Pomona.

He was a man of sterling character,—kindly, considerate, and polite. Notwithstanding his military training, he was always a peacemaker, and had many of the best traits of his Scottish ancestry. He had a remarkably erect and dignified bearing, strictly in keeping with his character. He was an honorable, upright Christian gentleman, and is greatly mourned by his old friends in Southern California.

Mr. Sanders was elected a Member of the American Society of Civil Engineers on February 3, 1904.

WALTER HOWARD SAWYER, M. Am. Soc. C. E.*

DIED DECEMBER 21, 1923.

Walter Howard Sawyer, the son of John Marshall and Laura James Sawyer, was born at Middletown, Conn., on May 21, 1867. Left fatherless in his youth, he worked his way through school and was graduated from Bromfield Academy in Harvard, Mass.

Mr. Sawyer studied engineering in the office of Shedd and Sawyer, of Boston, Mass., and in the evening schools of that city. From 1885 to 1902, he served consecutively as Leveler on the New York and Boston Rapid Transit Railroad, as Assistant Engineer with Mr. E. W. Bowditch, of Boston, as Assistant Engineer with Mr. Edward A. Buss, of Boston, as Resident Engineer on the hydro-electric development at Rumford Falls on the Androscoggin River, at Rumford, Me., with the Burlington Woolen Mill, Winooski, Vt., and as Assistant Engineer with Mr. Edward Sawyer, of Boston.

In February, 1902, Mr. Sawyer was appointed Agent of the Union Water Power Company, at Lewiston, Me., having responsible charge of the extensive storage system at the head-waters of the Androscoggin River and of the large water-power interests of that Company at Lewiston. In 1909 he was appointed Agent and Hydraulic Engineer of the Androscoggin Reservoir Company of Lewiston, a company formed for the purpose of developing additional storage at the head-waters of the Androscoggin River, in which connection he promoted and helped design and build the Aziscohos Dam located in Lincoln Plantation, which created a storage basin with a capacity of 9 500 000 000 cu. ft. The development of the resources of these two companies was Mr. Sawyer's great aim in life.

In private practice, he was a member of the firm of Sawyer and Moulton, Engineers, of Portland and Lewiston, Me., from 1909 to 1914, and of the firm of Sawyer and Bean, Consulting and Hydraulic Engineers, of Lewiston, from 1918 until his death.

On April 23, 1900, Mr. Sawyer was married to Helen F. Hayes, of Wellesley, Mass., who, with one daughter, Elizabeth Hayes Sawyer, and two sisters, survives him.

Mr. Sawyer was the inventor of an adjustable log sluice for high dams; of a method of measurement of water by electro-chemical means; and a method of sewage treatment; and the author of "Monograph on Ice Expansion."

He was widely consulted in water-power matters, especially water-storage problems, having been considered an authority on such questions. Professionally, he will be remembered best for his successful work in developing the storage resources and in regulating the flow of the Androscoggin River in Maine. By his skill, his knowledge, his faith in his projects, his tact and honest methods, he brought his visions to practical completion, giving to that section of Maine a prosperity which it had never known before.

In civic matters he served his home city of Auburn, Me., as Alderman and as member of the Water Board for two years, and was one of the leaders in

* Memoir prepared by P. L. Bean, M. Am. Soc. C. E.

the movement which gave the city its present City Manager form of government. He was responsible for the formation of the Auburn Sewerage District and served as a member of the Board.

During the World War he was an active member of the Maine Public Safety Committee, Secretary and Treasurer of the Androscoggin County Committee of Public Safety, and member of the Federal Milk Commission of New England for the State of Maine.

Mr. Sawyer was a practical man, a firm believer in private enterprise and hard work, and possessed of a high sense of honor and justice. His business associates, neighbors, and many young men struggling for an education, know the kind heart of the man whose keen judgment, pleasant word, and practical assistance made the day brighter and the path of life easier. His home was a delight to his friends; his love of flowers and children gave a glimpse of the splendid heart of the man. One who knew him in boyhood paid him this tribute:

"He was cheerful, earnest, thorough going all the way along; to know him was to appreciate the full measure of his manhood. The world is richer for his journey through it, the poorer for that journey's having come to a too early end".

He was a member of the Boston Society of Civil Engineers and of the New England Water Works Association, a Charter Member and Past-President of the Maine Society of Civil Engineers, a member of the Society of Municipal Improvements, and of numerous clubs. On June 7, 1920, the honorary degree of Master of Science was conferred on him by the University of Maine in recognition of his attainments in the engineering field.

Mr. Sawyer was elected a Member of the American Society of Civil Engineers on May 2, 1906.

CLINT SANFORD SLAYBACK, M. Am. Soc. C. E.*

DIED OCTOBER 26, 1922.

Clint Sanford Slayback was born at Princeton, Ind., on February 11, 1859, and attended the public schools of that city until he was sixteen years of age.

In 1876, he entered the employ of the Cincinnati Southern Railroad Company, as Axeman and Rodman. After two years of service with this Company, he accepted the position of Rodman and Instrumentman with the Louisville, New Albany and St. Louis Air Line, now the Southern Railway Company. From 1880 to 1881, he served as Resident Engineer of the Mt. Vernon Branch of the same railroad, and, during 1882, held the position of City Engineer of Mt. Vernon, Ind.

From 1882 to 1885, Mr. Slayback was Resident Engineer of the Evansville and Indianapolis Railroad, and, later, accepted the position of Engineer with the Evansville and Terre Haute Railroad Company, which he retained until 1887, when he entered private practice as a Consulting Engineer. In 1888, he became Resident Engineer of the Evansville and Richmond Railroad, and served in this capacity for three years, when he was made City Engineer of Anderson, Ind. From 1893 to 1897, he was again engaged in private practice as Consulting Engineer.

Mr. Slayback then entered the paving business as Engineer and Superintendent for the Talbot Paving Company at Shreveport, La. He held this position until 1898, doing pioneer work in paving, after which he accepted the appointment of Locating Engineer with the Northern Railroad Company, between Little Rock, Ark., and Springfield, Mo. In 1899, Mr. Slayback was made Engineer in Charge of the Chicago, Portland, and Southeastern Railroad and was engaged in promotion work, holding this position for one year. From 1900 to 1901, he served as Chief Engineer for the Searcy and Des Arc Railroad Company.

His next engagement was that of Resident and Locating Engineer of the Missouri, Kansas and Texas Railroad System, which position he resigned in 1903 to re-enter private practice. From 1905 to 1906, he acted as Chief Engineer for the White River and Mountain Home Railroad Company and from 1906 to 1911, as Office Engineer for the Union Railway Company, at Memphis, Tenn. During 1912, he resumed his private practice.

In 1913, Mr. Slayback was appointed Chief Engineer of the McNerney Company of Muskogee, Okla., but, in 1915 and 1916, was again engaged in private practice. In 1917, he became Engineer of the Talbot Engineering Company at Houston, Tex., and remained with this firm for one year, afterward accepting a position with the Engineering Department of the Texas Company of Houston and Cisco, Tex., in the Refinery and Production Departments. He was a popular and agreeable companion, and well liked by his co-workers.

Mr. Slayback was elected a Member of the American Society of Civil Engineers on March 7, 1921.

* Memoir prepared by J. H. Brillhart, M. Am. Soc. C. E.

HOWARD SOULE, M. Am. Soc. C. E.*

DIED DECEMBER 10, 1924.

Howard Soule was born on December 8, 1829, at Sennett, Cayuga County, N. Y. He received a common school and academic education at Auburn, N. Y.

In his early experience Mr. Soule was engaged in making preliminary surveys for railways from Auburn to Ithaca, N. Y., and from Auburn to Lake Ontario; later, in 1856, 1857, and 1858, he was in charge of the construction of the railroad from Auburn to Lake Ontario, which is now part of the Lehigh Valley System.

Between 1847 and 1874 he was engaged principally on State projects in connection with the enlargement of the Erie Canal in Central New York. He made surveys for draining the Montezuma Marshes at the foot of Cayuga Lake and was in charge of the work done from 1858 to 1861 in consummation of that project. In 1862 and 1863, he made the surveys for the proposed "Gun-Boat" locks on the Middle Division. He served the State in the positions of Assistant Engineer, Resident Engineer, and Division Engineer, of the Middle Division of the State Canals, ending his State service in 1874.

Mr. Soule served as City Engineer of Syracuse, N. Y., from 1874 to 1876. Afterward, for twelve years, he was engaged in the contracting business, designing and constructing iron bridges, of the Whipple, cast-iron, bow-string type, in general use for many years on the State Canals. At other times during this period he was engaged in the construction of seventy miles of the "West Shore" Railroad, in raising the banks and locks of the Welland Canal (in Canada), and in lengthening the locks of the Erie Canal.

In 1890, he was called on to act as Consulting Engineer and to direct the design for the installation of a supply of water to Syracuse from Skaneateles Lake. He continued in that capacity until the water commenced to flow through the first conduit on July 4, 1894.

Although in practical retirement after this time, Mr. Soule's counsel was often sought, and, in 1905, notwithstanding his advanced age, he again gave valuable service in the installation of the second conduit from Skaneateles Lake.

Howard Soule was a skillful engineer and retained his practical ability and keen common sense to the end of his days. Self-effacing, and retiring in disposition, he holds a high place in the esteem of those who were fortunate enough to know him.

Mr. Soule was elected a Member of the American Society of Civil Engineers on March 17, 1869.

* Memoir prepared by Henry C. Allen, M. Am. Soc. C. E.

FREDERICK PUTNAM SPALDING, M. Am. Soc. C. E.*

DIED SEPTEMBER 4, 1923.

Frederick Putnam Spalding, the son of Israel Putnam and Ruth Elizabeth (Cooley) Spalding, was born at Wysox, Pa., on April 7, 1857. He was of English ancestry, a descendant of Edward Spalding, who first settled at James Cittie, Va., in 1619 and removed to Braintree, Mass., in 1630. It is of interest to note that in 1663 Edward Spalding was "the Surveyor of Highways" at Chelmsford, Mass. Israel Putnam Spalding served as a Union soldier during the Civil War and participated in the Battles of Fredericksburg, Chancellorsville, and Gettysburg. He was mortally wounded at Gettysburg on July 2, 1863, and died twenty-six days later, when his son, Frederick, was only six years of age.

Frederick Putnam Spalding received his preliminary education in the public schools of Towanda, Pa. He then entered Lehigh University and was graduated with the Class of 1880 with the degree of Civil Engineer.

Immediately after his graduation, Mr. Spalding entered the employ of the Southern Pacific Railway Company as Levelman on preliminary and location surveys in Texas and New Mexico, resigning this position in June, 1882, to become Recorder and Observer on preliminary triangulation on the United States Geodetic Survey in the State of Pennsylvania. He remained at this work, however, only four months.

From October to December, 1882, he served as Transitman on the survey of Memphis Reach, Mississippi River; from January to March, 1883, he was in charge of hydraulic grading at Memphis Reach; and from July, 1883, to March, 1884, he was Assistant to the Engineer in charge of that work.

From April to August, 1884, Mr. Spalding was Assistant Engineer in charge of a field party on a survey of the Arkansas River, and from August, 1884, to March, 1885, he was Assistant to the Engineer in Charge, in direct charge of all the work of Memphis Reach and Harbor.

From February, 1885, until August, 1886, Mr. Spalding was engaged in private practice in Memphis, Tenn., under the firm name of Spalding and Burrowes. He was then called to his Alma Mater, Lehigh University, as Instructor in Civil Engineering, but resigned in October, 1888, to become Engineer in Charge of Surveys for the Third District of the Lower Mississippi, at Greenville, Miss. He relinquished this work in June, 1890, to become Engineer in Charge of Street Extensions in the District of Columbia, and in November, 1890, he was appointed Engineer in Charge of Rock Creek Park, Washington, D. C.

In October, 1891, Mr. Spalding was called to Cornell University, Ithaca, N. Y., as Assistant Professor of Civil Engineering and continued in this position for seven years, when he resigned to engage in business as a Contractor on municipal work in Bethlehem, Pa. He remained in this line of work only two years.

* Memoir prepared by A. Lincoln Hyde, M. Am. Soc. C. E.

In September, 1900, he went to the University of Missouri at Columbia, Mo., as Professor of Civil Engineering, and during the academic year 1905–06, he served as Junior Dean of the School of Engineering. He continued as Professor of Civil Engineering in the University of Missouri until his death on September 4, 1923.

Professor Spalding was married on January 22, 1885, to Annie Packer Wilbur, of Bethlehem, Pa. He is survived by his wife and two daughters, Marion Elizabeth and Aurelia, and by a brother, Israel Putnam Spalding, Jr., of Towanda.

Professor Spalding was a Fellow of the American Association for the Advancement of Science, a member of the Society for Testing Materials, the Society for the Promotion of Engineering Education, the Society of Sigma Xi, Tau Beta Pi, and the Faculty Club of Columbia, Mo.

He was the author of "Notes on Hydraulic Cement" (1893); "Text-Book on Roads and Pavements" (1894, 1901, 1908); "Hydraulic Cement, Its Properties, Testing, and Use" (1897, 1905); and "Masonry Structures" (1921).

While on his way back from a vacation pleasure trip to Southern California with his wife and two daughters, Professor Spalding was seized with an attack of heart failure in the railway station at San Francisco. The party was able to continue its trip, but he suffered a second attack at Portland, Ore. Following medical advice and accompanied by a trained nurse, the family continued on the journey home, but at St. Paul, Minn., it became necessary to remove Professor Spalding to St. Luke's Hospital. He rallied again, but died suddenly as he was chatting with his nurse about twelve hours after his arrival at St. Paul. He was buried at Bethlehem, Pa.

The following resolution was adopted by the Faculty of the School of Engineering of the University of Missouri:

"Frederick Putnam Spalding, for twenty-three years Professor of Civil Engineering in the University of Missouri, died suddenly just before the opening of the Fall Semester, 1923. In his death we have suffered the loss of one who was a recognized authority in his special field, a wise and patient teacher, a valued counselor to this faculty, a just, gentle, and beloved man. His memory will remain with us his colleagues, as it will with those who have gone out from his teaching into the engineering work of the world—an inspiration and a blessing."

Professor Spalding was elected a Member of the American Society of Civil Engineers on December 2, 1891.

JACOB STEPHEN SPIKER, M. Am. Soc. C. E.*

DIED APRIL 28, 1924.

Jacob Stephen Spiker, the son of John and Jane Spiker, was born in Clay County, Illinois, on June 5, 1861. He received his early education in the public schools of Clay County. After teaching for two years, he entered Vincennes University from which he was graduated in 1887.

On leaving the University, he was elected County Surveyor of Knox County, Indiana, and filled this office for three years, resigning to pursue further studies. Accordingly he entered Purdue University and took a special course in civil engineering, which he completed in 1893.

Returning to Vincennes, Ind., Mr. Spiker opened an office and engaged in private practice. He was also appointed Engineer of Bridges and Roads for Knox County, Indiana.

In 1897, he was elected City Engineer of Mt. Carmel, Ill., and designed and constructed street paving, sewers, and water-works. In the following year he was made Chief Engineer of Gills and Shaker Prairie Levee Districts in Sullivan and Knox Counties, Indiana, involving the design and construction of about 80 miles of levee work along the Wabash River.

From 1905 until 1916, Mr. Spiker was engaged in the general practice of engineering in Southern Illinois and Indiana and Southeastern Missouri. In 1916, he was appointed Chief Engineer of the Brevoort Levee System, in charge of its design and construction. He was also engaged as Engineer in the straightening of the Patoka River in Pike and Gibson Counties, Indiana.

From 1918 to 1922, Mr. Spiker served as City Engineer of Vincennes, and from 1922 until his death, was actively engaged in the general practice of engineering and as Chief Engineer of the Little Black River Drainage District in Southeastern Missouri. He also took a prominent part in construction work in Kentucky where he contracted his last sickness. Surviving him are his wife and one son.

Mr. Spiker was elected a Member of the American Society of Civil Engineers on February 4, 1914. He was also a member of the Western and Indiana Engineering Societies.

* Memoir prepared by A. C. Spiker, M. Am. Soc. C. E., and Noble Spiker, Esq.

EDWARD CHANDLER TOLLINGER, M. Am. Soc. C. E.*

DIED JUNE 18, 1924.

Edward Chandler Tollinger, the son of George Washington and Margaret Jane Tollinger, was born in Lancaster County, Pennsylvania, on September 20, 1844. He received his education in the schools of Lancaster County and attended the Pennsylvania State Normal School at Millersville, Pa. He served in the Union Army during the Civil War. After the war he began his professional career as a Rodman on railway work, and from then until his death, Mr. Tollinger was actively engaged in the practice of civil engineering.

From 1869 to 1873, he was with the Chicago and Alton Railroad Company, serving from June, 1869, to July, 1870, as Rodman and Instrumentman, and from 1870 to 1873, as Assistant Engineer in general charge of construction on 65 miles of line in Illinois and 102 miles in Missouri.

During 1873 and 1874, he constructed 5 miles of levee, on the Mississippi River (Sny Island Levee in Missouri). From July, 1875, to August, 1876, he was Assistant Engineer and Masonry Inspector on the St. Louis, Keokuk and Northern Railroad, and from 1877 to 1880, Superintendent of Construction and Assistant Engineer on the Muscle Shoals lock and dam construction on the Tennessee River, where he completed Locks Nos. 3 and 4 and started construction on Lock No. 2.

Mr. Tollinger held the position of General Manager and Superintendent for the Keokuk Hydraulic Mining Company at Keokuk, Iowa, from 1881 to 1883, and as such constructed a reservoir, flumes, approximately 14 miles of ditch, and other work connected with the project. While thus employed, he also served during the winter season in the Third Mississippi River Commission District on channel work and revetment construction.

In 1884, Mr. Tollinger accepted permanent employment with the Mississippi River Commission as Assistant Engineer, and from 1884 to 1886, was engaged on channel improvement and revetment work. In 1886, he was assigned as Assistant Engineer in charge of levee construction in Arkansas and Louisiana (Upper Tensas Levee District).

From 1886 until his death, he was actively in charge of levee construction and maintenance in the Third Mississippi River Commission District, covering one or both of the Levee Districts comprising 426 miles of levee in Mississippi, Arkansas and Louisana.

Although eligible for retirement, Mr. Tollinger, by recommendation of the various District Engineers and the members of the Mississippi River Commission, was retained in active service, and, until the beginning of his last illness, was relied on for expert administration and advice in the layout and conduct of all levee work performed by the Government in the Third District. His services were particularly valuable during times of flood emergency; he was always resourceful and never gave up the fight. To his credit more than to any other man is ascribed the successful maintenance of the levee lines throughout scores of high-water periods.

* Memoir prepared by A. M. Todd, M. Am. Soc. C. E.

Mr. Tollinger was very popular with all his associates and acquaintances, and he leaves a host of friends, not only in the territory in which he served and lived from day to day, but throughout the whole country. He was also well known, and many will miss his kindly and genial personality. He was most generous with his means and always could be relied on to help the deserving, but his modesty prevented many of his kindly acts from being generally known.

Mr. Tollinger is survived by three brothers, L. Tollinger and C. Tollinger of Delavan, Ill., and George Tollinger, of Pennsylvania, and by two sisters, Mrs. S. J. Elliott and Mrs. Leonard W. Lee, both of Oxford, Pa.

He was a member of the American Society of Military Engineers, the National Geographic Society, and the Benevolent and Protective Order of Elks. He was also a member of and Elder in the Presbyterian Church.

Mr. Tollinger was elected a Member of the American Society of Civil Engineers on June 4, 1902.

JOHN CRESSON TRAUTWINE, JR., M. Am. Soc. C. E.*

DIED JULY 4, 1924.

John Cresson Trautwine, Jr., the son of John Cresson and Eliza Ritter Trautwine, was born in Philadelphia, Pa., on March 17, 1850.

He was educated in private schools, and was trained in commercial work in the employ of a firm of iron brokers, Morris Wheeler and Company, and then in the service of his father whom he assisted for several years in the exacting and painstaking work of revising and editing the "Civil Engineer's Pocket-Book."

Collaborating with the late Rudolph Hering, M. Am. Soc. C. E., in 1887, Mr. Trautwine translated Ganguillet and Kutter's "Flow of Water in Channels". In 1890, with Arthur Marichel, he translated parts of H. Bazin's "Flow of Water over Weirs" and, in 1904, he collaborated with Olga and Boris Simin in the production of their paper on "Water Hammer".†

For twelve years he was Secretary of the Association of Engineering Societies and Editor of its *Journal,* and for four years held the position of Chief Engineer of the Bureau of Water of the City of Philadelphia. At intervals, from 1892 to 1907, he served that city on Civil Service Boards in the examinations of candidates for municipal engineering positions.

In his consulting practice, Mr. Trautwine covered a wide range of engineering activity, bringing to the service of his clients deep technical knowledge and sound practical judgment. Acting either individually or on commissions, he investigated, reported and advised on many engineering projects, including a water distillation plant at Pittsburgh, Pa.; the location of an electric car line in Northeast Philadelphia; the condition and utilization of various leased lines of the Mine Hill and Schuylkill Haven Railroad Company; and public water supplies in Southern New Jersey, Mount Sterling, Ky., Red Bank, N. J., and Cumberland, Md. In November, 1898, at the request of the General Committee on Water Supply of New York, N. Y., he advised respecting the further utilization of the Croton water-shed, and the necessity of developing other sources of supply, and, in 1909, he investigated the plans and prospects of the Rochester and Lake Ontario Water Company, Rochester, N. Y.

Mr. Trautwine was methodical in his habits and systematic in his business, studying efficiency and ways of minimizing the time and effort necessary to accomplishment, thus finding time outside his professional work to take a useful part in civic activities. His interest in the community life was constant and his views were always constructive.

He was of a genial disposition, interested in life and its philosophy, a close observer of events, and a shrewd practical interpreter of the developments of the times. He was broad-minded in his views of social customs and tendencies, tolerant of the idiosyncrasies of his fellows, and always kindly and helpfully disposed.

* Memoir prepared by Henry H. Quimby, M. Am. Soc. C. E.

† *Proceedings,* Am. Water Works Assoc., Vol. 24 (1904), p. 341.

Mr. Trautwine had a happy facility in public speaking, being fluent and clear, with his subject matter logically arranged. He was ready witted and convincing in argument; his contribution to the discussion of an engineering paper was always practical and illuminating; and he was quick to see and adept in pointing out any humorous reference in an otherwise dry scientific question.

Mr. Trautwine was a life member of the Franklin Institute, and served on its Committee of Science and the Arts, and on the Board of Managers. He was a member, former Secretary, and Past-President of the Engineers' Club of Philadelphia, a member of the American Society of Mechanical Engineers and of the American and the New England Water Works Associations, and an Associate of the Institution of Civil Engineers, London, England. He was also an active member of the City History Society of Philadelphia for which he had prepared several valuable papers, among the most painstaking of which was a history of the Old Columbia Railroad, illustrated with a number of photographs. He presented this paper in various places and it will be printed as a publication of that Society. He was also a member of the Historical Society of Pennsylvania and of the Philobiblon Club, and was President of the Economic Club for three years.

Mr. Trautwine's name frequently appeared in the Philadelphia *Public Ledger* signed to letters on many subjects, which he had written.

He was married on May 23, 1872, to Lucy L. Smith, who, with their son, John C. Trautwine, 3d, M. Am. Soc. C. E., survives him.

Mr. Trautwine was elected an Associate of the American Society of Civil Engineers on December 5, 1888, and a Member on April 26, 1921.

CHARLES HOPKINS VAN ORDEN, M. Am. Soc. C. E.*

DIED MAY 9, 1924.

Charles Hopkins Van Orden was born at Catskill, N. Y., on April 11, 1847. His father, William H. Van Orden, was a lineal descendent of Peter Schuyler, the first Mayor of Albany, N. Y. His mother, Mary (Hopkins) Van Orden, was descended from Stephen Hopkins, one of the signers of the Declaration of Independence.

From 1873 to 1896, Mr. Van Orden was with the United States Coast and Geodetic Survey and served, successively, as Aide, Sub-Assistant, and Assistant, principally in the South and in New England. He was Chief of Party for five seasons, Acting Chief for three seasons, and Senior Officer for six seasons. This work consisted principally of triangulation, but also included topographic, hydrographic, and State line work. During the greater part of the last seven years of this service, he had charge of the field work of the Massachusetts Topographical Survey Commission.

In addition, Mr. Van Orden ran a double line of levels, 335 miles, from Boston, Mass., to Dobbs Ferry, N. Y., by way of Albany, N. Y. He also established tide gauges on the eastern end of Long Island, in Buzzards Bay, in Block Island, and in the Bay of Fundy.

After his resignation from the U. S. Coast and Geodetic Survey in 1896, Mr. Van Orden was sent to determine the boundary line between Vermont and Canada, following which he engaged in private practice, as Consulting Engineer, at Catskill. In 1922, his health failed, and he was obliged to retire, continuing his residence in Catskill until his death on May 9, 1924.

His work as a Consulting Engineer included the construction of a freight railroad, 1 mile long, serving a shale brick factory, also a long, heavy trestle at a steep grade, used to transport shale to the top of a building by locomotive. He was also engaged on the construction of the sewerage system for Catskill and in general municipal engineering work. Mr. Van Orden also devised a plan for a water supply for Athens, N. Y., and was engaged as an expert to testify before a State Legislative Committee relative to the improvement of streams.

On July 16, 1890, Mr. Van Orden was married to Helen McLean, who, with a daughter, Mrs. A. J. De Gozzaldi, and a son William Wendel Van Orden, survives him.

He was an Elder of the Dutch Reformed Church of Catskill, and had been a member of the Holland Society of New York for a number of years.

Mr. Van Orden was elected a Member of the American Society of Civil Engineers on June 1, 1898.

* Memoir compiled from information on file at the Headquarters of the Society.

HENRY HAYES WADSWORTH, M. Am. Soc. C. E.*

DIED JULY 7, 1923.

Henry Hayes Wadsworth was born on January 13, 1865, at New Haven, Conn., in which State his forebears had resided for many generations. The founder of the Wadsworth family in America, William Wadsworth, landed at Salem, Mass., in 1632, from England, and soon afterward settled at Hartford, Conn. His son, Captain Joseph Wadsworth, was the chief actor in that well known story of the concealment, in the now famous Charter Oak, of the charter of the Connecticut Colony, when it had been demanded for revocation by Sir Edmund Andros, first Governor-General of New England. Henry Hayes Wadsworth was of the seventh generation in direct line from Joseph Wadsworth and of the eighth generation from the original settler. His mother, who was Fidelia Munn Gilbert, was the descendant of a long line of Gilberts who had lived in Connecticut since some time prior to 1660, when the first of the family served as Deputy Governor of the New Haven Colony.

In 1870, his father, Henry Wadsworth, moved to Minnesota with his family of three boys and one girl, and established a new home at Glencoe, a short distance west of Minneapolis. Here, Henry Hayes Wadsworth, the second son, received his common school education. Later, however, he returned to New Haven to take his college preparatory work and to attend the Sheffield Scientific School of Yale University. He completed the regular course in Civil Engineering, and was graduated with special honors in 1886, at the head of the Civil Engineering Section of his class.

After his graduation, Mr. Wadsworth returned to Minnesota, arriving at a time when pioneer railroad work was in full swing. For four years, he was engaged on the location and construction, in Minnesota and North Dakota, of lines which afterward became parts of the Great Northern System. The latter portion of this period he served as Assistant Engineer and Chief Draftsman of the Eastern Railway of Minnesota, during which time the elevated sections of that road in Duluth, Minn., were designed and built. This work included many special structures and complicated crossings. He then moved to the head of Lake Superior, where a rapidly developing harbor offered promise of a greater breadth and variety of opportunity. From 1891 until 1895, he was Principal Assistant City Engineer of Superior, Wis., in charge of all street, bridge, sewer, and harbor works. Among other improvements under his direction, about 30 miles of pavements were laid and 40 miles of sewers built. Following his municipal work at Superior, he was engaged by the Lake Superior and Ishpeming Railway Company, as Acting Chief Engineer. For about one year he had charge of the revision of its location and the building of its road from Ishpeming to Marquette, Mich., together with the design and construction of ore and merchandise docks for the railroad, in the harbor at Marquette.

In September, 1896, Mr. Wadsworth reported for duty at the United States Engineer Office in Duluth, which office has supervision of the improvement of the Duluth-Superior and other harbors on Lake Superior, and thus (except

* Memoir prepared by Lee S. Griswold and Ralph G. Wadsworth, Associate Members, Am. Soc. C. E.

a previous period of temporary employment during the summer of 1895) he began his service with the Federal Government, which lasted for more than twenty-three years. Soon after his appointment, he was promoted to the grade of U. S. Assistant Engineer and placed in immediate charge, at Superior, of dredging operations involving the removal of approximately 21 000 000 cu. yd. of material, about equally divided between the harbors at Duluth and Superior, carried on under a contract that continued for about five years. In addition to the dredging referred to, harbor developments under way included the dredging of other channels and basins, establishing beacons, locating docks, and the construction of breakwaters and jetties, together with the hydrographic surveys incident thereto. The harbor surveys were largely made in winter, when ordinary land-surveying methods could be used on the ice, soundings being taken through borings made by specially designed portable machines. On the dredging work, both hydraulic and dipper dredges of large capacity were used. Mr. Wadsworth was in direct control of much of this work. He also had charge of extensive additions to the training walls of the Duluth Ship Canal and prepared one of the early plans for the aerial bridge which was constructed later across the Canal.

In 1905, Mr. Wadsworth accepted a transfer to San Francisco, Calif., where he was appointed Principal Assistant Engineer for the California Débris Commission and Second San Francisco District Engineer Office. The Débris Commission is a Board of Army Engineer Officers which has control of hydraulic mining operations in California. The work of this Board has been described in a paper* presented to the Society by William W. Harts, Colonel, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., entitled, "The Control of Hydraulic Mining in California by the Federal Government." At the time of Mr. Wadsworth's arrival, a contract had just been let for the raising of the Yuba River Barrier, a restraining weir which formed one unit of the extensive project for river improvement described in the paper mentioned. Work on the Daguerre Point Cut and Settling Basin, the principal other feature of the project, was also under way. Supervision of the work on these and other portions of the plan formed a large part of Mr. Wadsworth's duties. The work done in connection with the Barrier, until its failure in 1907, was described by him in a paper† entitled "The Failure of the Yuba River Débris Barrier and the Efforts Made for Its Maintenance", presented to the Society in 1910.

The Secretary and Disbursing Officer of the Débris Commission serves also as District Engineer in charge of the control and improvement of the Sacramento and the San Joaquin Rivers with their tributaries in the central valley of California. Aside from actual construction work in connection with the restraint of mining débris, Mr. Wadsworth's duties included a wide range of studies and inspections throughout the central part of the State. Hydraulic mine inspections led to many remote places on the numerous tributaries of the Sacramento River and gave him a comprehensive grasp of stream-flow conditions in the Sierra Nevada Mountains. Surveys and examinations of the

* *Transactions*, Am. Soc. C. E., Vol. LVII (1906), p. 1.
† *Loc. cit.*, Vol. LXXI (1911), p. 217.

lower reaches of the rivers, to determine the effect of mining operations on navigation and agriculture, gave him an equally broad knowledge of valley conditions. Thus, he became eminently fitted to take up the problem of flood control of the Sacramento and San Joaquin Rivers, which problem occupied most of his attention during the period from 1908 to 1919.

In 1908, surveys were made under his immediate direction of the Sacramento River from Suisun Bay to the mouth of the Feather River and of the San Joaquin to Stockton, Calif. The resulting maps on a scale of 400 ft. to the inch covered more than 120 miles of river and included frequent cross-sections and a wide strip of adjacent topography. In 1909, the Sacramento River survey was extended about 120 miles farther up stream.* These extensive surveys were completed at about the same time that the first accurate measurements of severe floods on the Sacramento became available. In 1907 and 1909, floods occurred of greater magnitude than had ever previously been measured and they at once furnished the fundamental quantities on which studies of flood control had thereafter to be based. With these quantities in mind and the recently completed surveys before him, Mr. Wadsworth began studies looking to the formulation of a comprehensive flood-control plan for the Sacramento Valley. He soon demonstrated the unfeasibility of the channel enlargement plan, which had formerly been recommended, and then proceeded to a detailed design of a by-pass project, providing for the relief of the whole valley. His report, dated July 14, 1910, was embodied in the report of the California Débris Commission which was approved by the U. S. War Department and which has been the basis for the rapid and continuous development of the project since that time.

Immediately after the completion of his flood-control report, Mr. Wadsworth was transferred to the Department of the Interior and began an exhaustive study of all possible sources of water supply for the City of San Francisco, with a view to making recommendations as to granting to the City certain rights in the Hetch Hetchy Valley and Yosemite National Park. This investigation consumed a period of two and one-half years and resulted in the accumulation of a great mass of information, which was completely summarized and correlated in a report presented to the Advisory Board of Army Engineers in January, 1913.

Returning to duty with the War Department, a survey of the San Joaquin River, above Stockton, was made during the next two years, extending as far as Herndon, Fresno County, and including about 200 miles of river. The main purpose of this survey was to establish a basis on which plans for the control of floods in that valley could be developed, and partly to investigate the possibility of navigation of the river. The following years were occupied with studies and surveys of various phases of the flood-control projects and with certain construction work which had been undertaken by the Federal Government.

In the spring of 1917, Mr. Wadsworth applied for and received a commission as Major of Engineers in the U. S. Reserve Corps and, in May, spent eleven days at the First Officers' Training Camp at the Presidio of San Fran-

* These surveys were published as Documents Nos. 1123 and 1124, 60th Cong. 2d Sess., and Document No. 76, 62d Cong. 1st Sess.

cisco, before being relieved to resume his more pressing civil duties with the Engineer Department. In July, 1917, he was again on active duty for a short period at Salt Lake City, Utah, where he served as Senior Officer on a Board appointed to examine applicants for commissions in the Engineer Reserve Corps.

In January, 1920, Mr. Wadsworth resigned from the Government service to open an office in San Francisco, for consulting and general engineering practice, specializing in works for the utilization of water and for protection against floods. The announcement of his entrance into the ranks of consulting engineers brought immediate response from many quarters. He was engaged to make numerous examinations and reports on dams, irrigation and water supply projects, power development schemes, reclamation works, hydraulic mine projects, and was retained in the famous Antioch suit, which was directed against practically all the water users in the central valley of California. During this period, he also managed to devote a large amount of time to activities outside his engineering practice. As Chairman of the Section on Harbor Control of the Commonwealth Club of California—a public welfare organization of State-wide extent—he gave a great deal of effort to investigations and reports on various matters pertaining to the commercial development of San Francisco Bay and Harbor. He also served as Chairman of a Committee which reported to the Oakland, Calif., Chamber of Commerce on the water supply for the cities on the eastern side of San Francisco Bay just prior to the formation of the East Bay Public Utility District.

Mr. Wadsworth died in San Francisco on July 7, 1923, after an illness of only a few months. He is survived by two sons, Ralph G. Wadsworth, Assoc. M. Am. Soc. C. E., and Harold A. Wadsworth, Assistant Professor in the Department of Agriculture, University of California. His wife, who was Jennie M. Anderson, of North Dakota, and formerly of Ontario, Canada, died in 1919.

Mr. Wadsworth was a member of the Engineers' Club of San Francisco, the Commonwealth Club of California, the Home Club, of Oakland, Calif., the American Legion, the Society of American Military Engineers, and the Pacific Association of Consulting Engineers. He was a frequent attendant at the First Congregational Church of Oakland, the city in which he had made his home from the time of his first arrival in California.

He was of a modest but genial and kindly disposition, ever courteous and patient, yet energetic and forceful in completing whatever was undertaken, thorough, eminently just, possessed of high ideals, and of absolute integrity. He delighted in association with his friends and acquaintances, both professional and social, and held the respect, esteem, and confidence of all who knew him.

Mr. Wadsworth was elected a Member of the American Society of Civil Engineers on October 2, 1901. He took an active interest in the Society at all times and in addition to the paper previously mentioned, contributed numerous discussions on a wide variety of subjects to its *Transactions*. At the time of his death, he was Vice-President of the San Francisco Section.

GEORGE CHANDLER WHIPPLE, M. Am. Soc. C. E.*

DIED NOVEMBER 27, 1924.

The profession of Sanitary Engineering lost one of its most brilliant leaders in the death of Professor Whipple on November 27, 1924. Although he was thoroughly trained for his life's work, like many others, he was distinguished particularly for his powers of concentration, his unswerving devotion to his chosen profession, and the variety and extent of his accomplishments.

George Chandler Whipple was born at New Boston, N. H., on March 2, 1866. He was graduated from the Massachusetts Institute of Technology in 1889. In his Senior year at the Institute, he was a pupil of Professor W. T. Sedgwick from whom he received some of the inspiration for sanitary study and research which characterized his subsequent career. At that time, a force of well-trained men was being organized at Chestnut Hill Reservoir, a distributing reservoir within the city limits, to ascertain the reasons for some of the bad tastes in the water supplies of Boston, Mass. Mr. Whipple was one of the Principal Assistants in this work. He was employed in microscopical and other investigations and soon showed the results of his fine training. He developed an originality for designing apparatus to aid in the work and a faculty for perseverance in his researches which required little stimulus.

It was at this time that he invented many improvements in the methods of collecting and examining samples from different sources of supply and at different depths. This work led finally to the publication of his masterly book on "The Microscopy of Drinking Water" which went through several editions and was re-written and enlarged from time to time.

This remarkable book is perhaps one of the best among the monuments that remain to his memory. It gives an idea of the care and skill which he brought to bear on the many problems submitted to him for solution. It discusses methods of collection of samples for microscopical examinations; organisms found in lakes and reservoirs and their effects on odors, tastes, etc.; and covers questions connected with different methods of construction of reservoirs and their influence on the quality of supplies. In addition to this classical work, Professor Whipple published other books on such matters as "The Value of Pure Water" and "Typhoid Fever—Its Causation, Transmission, and Prevention".

In a comparatively short time, he rose to an important position among the sanitarians of the United States and became associated in business with Allen Hazen, M. Am. Soc. C. E., also a leading sanitary engineer in New York, N. Y., the firm name Hazen and Whipple, being distinguished in the practice of Sanitary Engineering.

* Memoir prepared by Desmond FitzGerald, Past-President and Hon. M., Am. Soc. C. E.

In 1911, Professor Whipple was honored by Harvard University, where a Chair of Sanitary Engineering was created for him and he became Gordon McKay Professor of Sanitary Engineering there and at the Massachusetts Institute of Technology.

In 1917, there appeared a summary of the public health movement in Massachusetts in three volumes, the result of his indefatigable zeal.

Professor C. E. A. Winslow, himself a distinguished sanitarian, has added the following brief summary of Professor Whipple's life work, which expresses a sympathetic appreciation of his accomplishments. He writes:

"Whipple conferred upon the discussion of the most technical problem a breadth of vision and a background of culture which gave a unique quality to all that he did. The untimely end of his life is a heavy blow to us all; but it was a life rich in useful service and full of the inner serenity and the outward helpfulness of a soul at peace with itself and in key with universal law."

He was married to Mary E. Rayner, of Chelsea, Mass., in 1893, and soon thereafter chose Cambridge as his permanent residence.

Professor Whipple was elected an Associate Member of the American Society of Civil Engineers on September 6, 1899, and a Member on October 6, 1908.

BENJAMIN EMANUEL WINSLOW, M. Am. Soc. C. E.*

DIED NOVEMBER 24, 1924.

Benjamin Emanuel Winslow, the son of Christian Wilhelm and Christiane Winslow, was born in Chicago, Ill., on July 2, 1867. After the Chicago fire of 1871, his parents removed to Richmond County, Virginia, where he spent his boyhood and was educated. In 1878 his family went to Denmark, and his education was continued at Copenhagen where from 1878 to 1883, he attended Hauck's Latin og Real Skole, from 1883 to 1888, the School of the Technical Society, and from 1889 to 1891, the Royal Academy of Fine Arts.

Mr. Winslow then returned to the United States, and on April 9, 1897, was appointed Architectural Engineer under the Commissioner of Buildings, City of Chicago, where he continued until 1906. From that date to 1915, he was employed as Architectural Engineer under several architects and engineers, notably Holabird and Roche of Chicago. In 1915, he was re-appointed to his former position as Architectural Engineer under the Commissioner of Buildings, City of Chicago, passing on structural plans submitted by applicants for building permits. He remained in this position until his death, except for a brief period, 1917-19, when he was assigned as Architectural Engineer in the Architect's Office of the Board of Education, Chicago.

Mr. Winslow was the inventor of the "Winslow Slide-Rule" and the author of the "Winslow Tables" for calculating the strength of wood, steel, and iron beams and columns. He was an engineer of ability and a man of exceptionally high standards of integrity and professional conduct, respected and esteemed by all who knew him.

He was married on October 18, 1908, to Hilfred Rudbeck, who, with one daughter, Gerda Marion, survives him.

Mr. Winslow was elected a Member of the American Society of Civil Engineers on October 2, 1907. He was also a member of the American Institute of Architects.

* Memoir prepared by T. L. Condron, M. Am. Soc. C. E.

COLIN REED WISE, M. Am. Soc. C. E.*

DIED MAY 19, 1924.

Colin Reed Wise, the son of Uriah Winfield and Henrietta Maria (Post) Wise, was born in Washington, Pa., on November 28, 1845. He was prepared for college under the direction of his father who was Professor of Greek and Latin in Mercer University, Penfield, Ga., where the family lived from 1855 to 1865. He entered Mercer University in September, 1861, but was obliged to leave in November, 1863, or be conscripted into the Confederate Army. He was on detail duty until the close of the Civil War, in hospitals at Atlanta and Macon, Ga. In January, 1866, Mr. Wise returned to college where he remained until the end of his Junior year. He then entered Dartmouth College from which he was graduated in July, 1868.

Immediately after his graduation, Mr. Wise was given charge of a school at Dividing Creek, N. J., and later taught at Newport, N. J. His health not being of the best, he resigned his position and, in 1869, joined a corps of engineers which was making surveys for a railroad in Northern New Jersey. The following year he was appointed an Assistant Engineer on the location and construction of the Northern Pacific Railroad in Minnesota.

On the completion of his work in Minnesota, in 1871, Mr. Wise returned to the East and was made Resident Engineer of the Hackensack and New York Extension Railroad, in Rockland County, New York; in 1873, he was appointed Chief Engineer of the railroad. On its completion in 1875, he entered the service of the New York, Lake Erie and Western Railroad Company and was engaged in the preparation of a general inventory of the property of that road. In 1877, he was appointed Assistant Engineer in connection with the construction of the New York Elevated Railroads in Greenwich Street and in Third and Ninth Avenues, New York, N. Y.

On August 18, 1880, Mr. Wise began the work of locating the New York, West Shore, and Buffalo Railroad, later having had charge of the construction of the Bergen County Division. Soon after the construction was finished, he was employed by the contracting firm that built the West Point Tunnel and the section of the railroad adjacent thereto, to prepare a statement of the work done by the firm, which statement resulted in the construction company paying the contractors a considerable sum of money as extras.

Mr. Wise was then sent to Georgia by Messrs. J. and W. Seligman and Company to inspect and appraise the Brunswick and Albany Railroad and other railroad properties in that State. On the re-organization of the Brunswick and Albany Railroad Company as the Brunswick and Western Railroad Company, he was made Chief Engineer and Superintendent of Construction. Afterward, as Assistant General Manager, he had charge of road maintenance and for a short time acted as General Manager. Messrs. Seligman and Company then sent Mr. Wise to Indian Territory to engage in work on an extension of the St. Louis and San Francisco Railroad to Paris, Tex., and, subsequently,

* Memoir prepared by R. S. Wise, Assoc. M. Am. Soc. C. E.

he was appointed General Engineering Agent in charge of the construction of a branch line from Wichita to Ellsworth, Kans.

During 1888, Mr. Wise was engaged in an appraisal of the construction work done by the contractor on the Washington Bridge across the Harlem River, New York, and on the sewerage and water supply systems of the Hackensack Water Company of New Jersey.

In 1889, he formed a partnership with Robert M. Watson, M. Am. Soc. C. E., and was engaged in municipal engineering work, sewer design, sewage disposal, water supply, etc. The firm designed sewerage systems for Liberty, N. Y., Rutherford, East Rutherford, Carlstadt, Oradell, Millville, Madison, and Garfield, N. J., and also prepared plans for sewage disposal plants for six of these towns, as well as plans for sewage disposal systems for a large number of manufacturing establishments. The firm also planned and superintended the construction of water-works systems for Wallington, Garfield, Hawthorne, North Arlington, Kingsland, and Lyndhurst, N. J., and Pearl River, N. Y. In addition, it prepared the plans for, and had charge of the construction of, the Paterson-Passaic-Hackensack Electric Railroad, the Paterson-Passaic-Hoboken Electric Railroad, and the Passaic-Newark Electric Railroad; designed all car barns, bridges, etc.; prepared plans for a trunk sewer and sewage disposal plant for Passaic and Bergen Counties, at the request of the Passaic Valley Sewerage Commission, which plans however were abandoned; and constructed the present Passaic Valley Trunk Sewer from Paterson to Newark, N. J. Mr. Wise was employed by the Commission to make all surveys in Bergen and Passaic Counties and the investigations, etc., in relation to the trunk sewer.

He served as City Engineer for Passaic, N. J., from 1892 until his death on May 19, 1924, with the exception of three years, when he was County Engineer for the County of Passaic. He was also Engineer for Acquackanonck Township, New Jersey, for 12 years, for Garfield, N. J., for 14 years, and for Wallington, N. J., for 10 years. During this time his firm (Wise and Watson) had charge of the building of several steel and concrete bridges across the Passaic River, reinforced concrete culverts throughout Passaic County, storm-water drains, undergrade crossings, and was engaged in general municipal engineering work.

In 1875, Mr. Wise was married to Serena Sherwood, of Spring Valley, N. Y. Mrs. Wise died on September 2, 1903. On December 8, 1904, Mr. Wise was married to Mrs. Lizzie Hutton, of Nanuet, N. Y., who, with two daughters and a son by his first marriage, survives him.

Mr. Wise had been prominent socially, professionally, and politically in Passaic for more than half a century. He was known throughout the State of New Jersey as a leader in his profession and was widely consulted in relation to engineering projects in the East. Aside from his professional work, Mr. Wise was active in the civic affairs of the community in which he lived. He was interested in athletics and was a member of the old Passaic Rod and Gun Club.

Mr. Wise was elected a Member of the American Society of Civil Engineers on April 6, 1904.

FREDERICK CHARLES YOUNG, M. Am. Soc. C. E.*

DIED MARCH 18, 1924.

Frederick Charles Young, the son of George N. and Mary Hunter Young, was born in Woodbine, Iowa, on June 13, 1886. He was prepared for college at the Woodbine Normal School, from which he went as honor scholar to the College of Applied Science of the State University of Iowa in the fall of 1906. He was graduated in June, 1909, completing the four-year course in three years, earning a large part of his own support, and winning three marked distinctions, namely, election to the Society of Sigma Xi, to Tau Beta Pi, Honorary Engineering Society, and the Cox Prize of $100 for the best graduating thesis. On graduation, he received the degree, Bachelor of Engineering, and in 1913 was given the degree, Civil Engineer.

During his summer vacations Mr. Young engaged in engineering work and immediately after his graduation assumed the position of Assistant City Engineer of Iowa City, Iowa, in charge of the construction of sewers and paving.

From September, 1909, to February, 1914, Mr. Young was an Instructor in Civil Engineering in the State University of Iowa, engaging in engineering work during his summer vacations. In June, 1913, he was given a contract to build a tunnel, ½ mile long, for the heating system of the State University of Iowa. He was so successful with this work that he determined to engage in contracting, and left the University in February, 1914, to begin this occupation in which he continued until September, 1917, when he entered active service in the United States Army, having been commissioned a Captain in the Engineer Officers' Reserve Corps, on August 8, 1917.

He reported for duty at the Second Engineer Officers' Training School, Fort Leavenworth, Kansas, on September 25, 1917. He was assigned to the 114th Engineers, 39th Division, Camp Beauregard, Louisiana, on December 10, 1917, and was appointed Regimental Adjutant, on March 2, 1918. He sailed for France on August 2, 1918, on the U. S. S. *Wilhelmina,* arriving at Brest on September 3. On October 3, he was placed in command of the First Battalion, 114th Engineers, and ordered to the front in the Meuse-Argonne engagement, where he was in charge of the maintenance of one of the three roads supplying this area until the signing of the Armistice. Captain Young sailed for the United States on April 1, 1919, on the U. S. S. *Nebraska,* arriving at Newport News, Va., on May 2, and was honorably discharged at Camp Dodge, Iowa, on May 30, 1919. The endorsement on his service record by the commanding officer was: "An engineer of marked ability, a careful administrator, and a capable executive".

He immediately resumed his contracting work with the Moore-Young Construction Company, in which his partner was Mr. Stanley D. Moore, of Waterloo, Iowa. The firm was engaged principally in the construction of municipal works, and Captain Young continued in this work until his death.

* Memoir prepared by William G. Raymond, M. Am. Soc. C. E.

He was married to Helen S. Ordway on July 22, 1908, at Glens Falls, N. Y. Two children, twins, were born to them, one of whom, Frederick Charles, Junior, died in infancy. The other, Gerald Ordway, and Mrs. Young are living.

Although Captain Young was a man of most persistent energy and attention to duty, as indicated by the record of his college work, yet he found time in the latter years of his more than usually successful business life for recreation and participation in public affairs. He was a lover of horses, and he and his wife and son found pleasure and health in riding the fine horses that he owned.

He was a member of the Rotary Club of Waterloo, and just before his death had been selected by the Committee on Nomination for Eldership in the Westminster Presbyterian Church of Waterloo.

A progressive and able executive, seeking and adopting the most advanced approved methods of conducting work, absolutely honest and honorable in all his dealings, Captain Young was one among the very small group of men who are able to build successful businesses in a few years and win and retain the respect and even affection of all those who know them well.

Captain Young was elected an Associate Member of the American Society of Civil Engineers on February 4, 1914, and a Member on August 12, 1920.

WILLIAM STUART BABCOCK, Assoc. M. Am. Soc. C. E.*

DIED JULY 17, 1924.

William Stuart Babcock, the son of Charles B. and Amelia (Hopkins) Babcock, was born at Evansville, Ind., on December 10, 1862 He received his technical education at the Massachusetts Institute of Technology where he specialized in Architecture and from which he was graduated in 1883.

From 1883 to 1884, Mr. Babcock was associated with Reid Brothers, Architects, at Evansville, and in 1884 and 1885 he was employed in the office of the Supervising Architect in Washington, D. C.

In 1886, he entered the service of the New York, New Haven and Hartford Railroad Company, in charge of the design and general superintendence of stations, shops, roundhouses, offices, and freight houses.

From 1886 until 1904, when he resigned his position with the Railroad Company, Mr. Babcock designed passenger stations, engine and freight houses in various cities and towns in Massachusetts, Rhode Island, Connecticut, and New York. He also made changes in the Company's Pier 19 at New York, N. Y., and designed an icing plant at Poquonock, Conn.

In 1904, he entered the service of the Barrett Company, of New York, as Special Water-Proofing Engineer, which position he retained until 1923, when he removed to Los Angeles, Calif. Being in ill health, however, he was unable to engage in business. He died at his new home on July 17, 1924.

Mr. Babcock was married at Huntington, N. Y., on October 26, 1907, to Louise Knapp Kane, who died in 1915. Two sons, Stuart Kane and Frederick Lloyd Babcock, survive him.

He was a member of the American Society for Testing Materials, the Engineers Club, the Technology Club, and the Railroad Club of New York. He was also a member of the Protestant Episcopal Church and, in politics, was a Republican.

Mr. Babcock was elected an Associate Member of the American Society of Civil Engineers on February 1, 1905.

* Memoir compiled from information on file at the Headquarters of the Society.

JUAN BATISTE HIPOLYTE BARDURY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 20, 1923.

Juan Batiste Hipolyte Bardury, was born at San Pierre, Martinique, on October 28, 1875. He received his technical education in France, where he was graduated in July, 1892, from the Rouan Professional School, with the degree "Contre Maitre."

Mr. Bardury followed various lines of work until September, 1896, when he was employed as Draftsman in the Compagnie des Mines at Chemin de Fer de la Guayanne Francaise. Six months later, he became Instrumentman in the same Company, which position he held until April, 1898. He then went to Havana, Cuba, and accepted a position with Poaling and Company, Limited, of London, England, as Inspector on the construction of the Malecon in the Harbor of Havana, which position he held until October, 1898. From that date, until April, 1899, he served as Private Secretary to the late General Frederick D. Grant, U. S. A., in Cuba.

From April, 1899, to July, 1902, Mr. Bardury was Draftsman and Instrumentman on the survey and construction of the new roads and bridges made by the Bureau of Public Works of Porto Rico, and, from August, 1902, until December, 1904, he held the position of Assistant Chief Engineer of the American Railroad Company of Porto Rico, on the survey and construction of the line from Mayagüez to Yauco.

From June, 1905, until March, 1906, he was engaged in the private practice of engineering in Porto Rico, during which time he drew the plans of the City of San German, Porto Rico, and constructed the market building for that city. From August, 1906, until February, 1907, Mr. Bardury served as Superintendent on the construction of 20 km. of railroad for the Fajardo Development Company, of Porto Rico, and from February to December, 1907, he was Assistant Superintendent on the construction of the Power Plant, Dam, and Tunnel at Comeria Falls, Porto Rico, for the J. G. White Company.

From January, 1908, to May, 1909, he was Assistant Engineer with the Bureau of Public Works, Porto Rico, in charge of the construction of the "Reyes Católicos" Bridge, a steel structure 100 m. long, and also of a 22-km. section of road construction from Lares to Yauco. From June until July, 1909, he was Draftsman in the Porto Rican Irrigation Service, and from July, 1909, to November, 1914, he held, successively, the positions of Assistant Engineer, Assistant Superintendent, and Superintendent of Construction, in the Porto Rican Irrigation Service on the surveys and construction of the Carite Dam and Tunnel.

From December, 1914, to the summer of 1915, Mr. Bardury visited his brother in Martinique. The brothers were the sole surviving members of their family, the others having been lost during the eruption of Mount Pelée in 1902.

* Memoir prepared by John M. Adams, Assoc. M. Am. Soc. C. E.

In July, 1915, Mr. Bardury became Assistant Engineer with the Department of Public Works of the Dominican Republic, and was employed during the next five years on the location and construction of highways. In December, 1920, he was appointed District Engineer of the Southern District, which position he held at the time of his death.

Mr. Bardury was a man of sterling character and marked ability and showed an earnestness and energy in everything which he undertook that inspired the confidence and esteem of all with whom he came in contact.

He was married on March 2, 1918, to Señorita Gloria Sanchez who, with his brother, survives him.

Mr. Bardury was elected an Associate Member of the American Society of Civil Engineers on April 2, 1913.

ERNEST DRINKWATER, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 30, 1924.

Ernest Drinkwater was born in Manchester, England, on January 22, 1869. Both his father, Thomas Holbrook Drinkwater, and his mother, Lucy (Dutton) Drinkwater, came of old Cheshire stock, tracing their descent to a period before the Norman Conquest.

He received his education in the elementary and technical schools of his native city, and in 1887 apprenticed himself to his father to learn the business of general contracting. At the termination of his apprenticeship, he became Assistant Engineer for his father's firm, and in August, 1896, assumed control of it, conducting the business successfully until 1902. Mr. Drinkwater had a natural talent for municipal work, and most of the firm's operations were along the lines of estate development, paving, sewerage and water supply, and housing.

In 1902, or early in 1903, he became convinced that better opportunities for professional advancement existed in the Dominions than in the "Old Country." Accordingly he emigrated to Canada (as did many other young Englishmen of his generation), settling in Montreal, Que., in the vicinity of which he spent the remainder of his life.

From May, 1903, to June, 1909, Mr. Drinkwater was in the employ of the Montreal Tramways, first as Construction Foreman, then, successively, as Assistant to the General Construction Superintendent, Inspector on power-house and sub-station construction, Storage Battery Engineer, and in charge of electrolysis surveys.

In July, 1909, he was appointed Superintendent of Construction and Chief Inspector of Works for the City of Westmount, a Montreal suburban municipality, a position which he held until June, 1912. The population of Montreal was increasing by leaps and bounds at that time, which increase required the expenditure of much more than the average amount of funds in street paving, sewers, etc.

On the termination of his Westmount engagement, Mr. Drinkwater became Engineer of the Town (now City) of St. Lambert, Que., just then beginning to shake off the lethargy of years of non-activity. He found it a quiet country village, with nothing in the way of modern municipal improvements save a water supply and sewerage system of sorts. He left it an up-to-date suburban residential city, doubled in population and lacking in nothing required for the health, comfort, or convenience of its citizens. During his comparatively short tenure of office, practically all settled streets were permanently paved and provided with sidewalks, the water supply and sewers were extended to care for the growing population, alterations and additions were made to various municipal buildings, the street lighting system was completely overhauled and modernized, and a number of parks were acquired, graded, and planted.

* Memoir prepared by R. De L. French, M. Am. Soc. C. E.

In December, 1916, Mr. Drinkwater opened an office in Montreal as a Consulting Municipal Engineer, which he maintained until 1920, when he was again called to St. Lambert as City Engineer, serving until 1923. While practicing independently, he designed and constructed the water supply and sewerage systems for the Town of Montreal South. In 1923, he again took up consulting practice, in which he continued until his death.

Among Mr. Drinkwater's tasks, during his second term of office in St. Lambert, was directing the expenditure of about $750 000 under the Provincial Housing Scheme. Upward of one hundred and fifty moderately priced residences were built, which materially helped to relieve the shortage of dwellings due to war conditions. It has been stated that this was one of the few projects under the Provincial Law to be carried out with entire success, as that Law did not work as well as its promoters hoped.

One of Mr. Drinkwater's outstanding characteristics was his ability to find time, despite his many professional duties, to take an active part in any social or charitable movement. Thus, he served as Honorary Secretary of the St. Lambert Soldiers' Memorial Committee, Honorary Treasurer of the St. Lambert Club, Vice-President of the Canadian Christmas Tree League, a Life Member of the Canadian Red Cross Society, and a Member of the Canadian Club of Montreal, the St. Lambert Players' Club, the St. Lambert Operatic Society, the Navy League of Canada, the Aerial League of Canada, the Royal Arcanum, the Sons of England Benefit Society, and St. Lambert Lodge, A. F. and A. M.

Mr. Drinkwater was also a member of the American Water Works Association, the American Society of Municipal Improvements, the American Public Health Association, the Royal Institute of Public Health, The Town Planning Institute of Canada, the South Shore Joint Town Planning Board, the Canadian Good Roads Association, and a Fellow of the American Geographical Society.

Having been something of an athlete in his younger days—he had won a number of medals at cross-country running—he never lost his interest in sport, although too busy a man to devote much time to it himself; he claimed that keeping up with his profession gave him sufficient exercise. He was a member of the Country Club of Montreal, the Royal Automobile Club of Canada, the St. Lambert Boating Club, and the Canadian Canoe Association.

In 1904, Mr. Drinkwater was married to Jennie Moores, of Manchester, England, who, with three children, Eric, Mollie, and Hereward, survives him.

Mr. Drinkwater was a man of strong opinions and principles, and had no hesitation in fighting for what he thought to be right. Although he had multifarious interests, he managed to keep in touch with the affairs of all the organizations of which he was a member, and could be thoroughly depended on in a crisis. He will be sadly missed by his professional brethren, and his death leaves a gap which those associated with him in social and charitable work will find hard to fill.

Mr. Drinkwater was elected an Associate Member of the American Society of Civil Engineers on September 11, 1917.

JOHN WARREN Du BOIS GOULD, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 20, 1924.

John Warren Du Bois Gould was born at White Plains, N. Y., on July 29, 1881. He was the son of the late E. Sherman Gould, M. Am. Soc. C. E., a well known engineer, and a great-grandson of Judge James Gould who founded, in a log house at Litchfield, Conn., the first law school in America. Mr. Gould's earlier education was obtained in the public schools of Yonkers, N. Y., in the St. John's Military School, of Manlius, N. Y., and in the Dwight School of New York City. In 1898, he entered the College of Engineering of New York University, from which he was graduated in 1902, with the degree of Bachelor of Science. Three years later, in 1905, following experience in the field, he earned the degree of Civil Engineer. He was a member of the Psi Upsilon fraternity, and active in behalf of his Alma Mater both as an undergraduate and as an alumnus.

Immediately after his graduation, Mr. Gould went to the Far West and engaged in railroad surveys and maintenance, and, later, in construction work for the Bay Cities Water Company, of San Francisco, Calif., especially in connection with the problem of additional water supply. In 1904, he became Assistant Engineer for the United States Reclamation Service in building the Gunnison Tunnel at Montrose, Colo. From 1904 to 1906, he was associated in New York, N. Y., with the Committee on Fire Prevention of the National Board of Fire Underwriters, as Engineer in charge of reports on the municipal water supply and distribution systems of important cities, including New York and Brooklyn, N. Y. From August to December, 1906, he was engaged as Principal Assistant Engineer of the Passaic River Flood District Commission, preparing surface estimates and reports on a proposed system of flood control for Paterson and Passaic, N. J., to be completed at an estimated cost of $3 000 000. This report was presented to the State Legislature of New Jersey in 1925.

From 1907 to 1911 Mr. Gould was connected with the firm of Gunn, Richards and Company. Previous to this, his work had been mainly in the technical problems of engineering, but from this time on, his experience became such as to require, in addition to technical qualifications, executive ability, the power of handling men, and of dealing with individuals representing important interests. During this period he undertook investigations and made reports on various industrial properties and on engineering and other projects, both for private corporations and for the U. S. Department of the Interior, under Secretary Garfield. He was then appointed Special Indian Inspector, and did much useful work for the Government, on the basis of his inspection of Indian Reservations in Colorado and other Western States.

In 1911, Mr. Gould entered private engineering practice with an office in New York, and was engaged from that time until his death in problems of

* Memoir compiled by Dean A. L. Bouton, Coll. of Arts and Science, New York Univ., New York, N. Y.

organization and re-organization. In 1912, he went abroad to carry on negotiations in behalf of the Indian Refining Company. From 1912 to 1914, he was engaged in closing the automobile manufacturing activities of the American Locomotive Company. In 1915-16, he visited Haiti and prepared an important general report on conditions in the island, in behalf of Messrs. Breed, Elliott and Harrison, of Cincinnati, Ohio, and Messrs. P. W. Chapman and Company, of Chicago, Ill., and New York. His report, made as a Special Representative in connection with the Chartered Company of Haiti, was of great value to the American Government and received the especial approbation of President Roosevelt.

During the World War, from July, 1917, until 1919, he was employed with the Food Administration as an Executive Assistant under Herbert Hoover, Hon. M. Am. Soc. C. E., and after the Armistice was in charge of the demobilization of the great organization which war conditions had brought into being.

For almost three years, beginning in 1919, Mr. Gould was at the head of the Industrial Department of Messrs. Bonbright and Company, but, in 1922, he returned to private engineering practice, in which he continued until November, 1923, when he accepted the large responsibilities of the General Managership of the H. H. Franklin Manufacturing Company, of Syracuse, N. Y., a position in which he continued until his resignation on February 1, 1924, on account of illness.

Mr. Gould's career was cut off when it was still incomplete and apparently at the moment when its supreme fruition in achievement was just beginning, but not before the qualities of a rare character were fully defined. Thoroughness in every detail; profoundly sympathetic but accurate judgment of men; an imaginative grasp of great projects; unusual powers of organization and of execution in large business problems, together with the ability to secure the loyal co-operation of subordinates; vital intensity of devotion to any responsibility which he had once accepted, joined with a fine capacity for intellectual detachment, by which he was able to preserve his sense of proportion; modesty and great generosity of spirit; and fearless and inflexible integrity in behalf of the highest business and personal standards, all these qualities make an unusual personal heritage to his profession, and a rich memory of unforgettable inspiration to his family, and to all who had the privilege of his personal friendship.

He died suddenly at his summer home, in Sharon, Conn., of a thrombus in the heart, on September 20, 1924, and is survived by his wife, Evelyn L. Fisk, daughter of the late Harvey Fisk, and two children, John and Evelyn.

Mr. Gould was elected a Junior of the American Society of Civil Engineers on October 7, 1902, and an Associate Member on October 1, 1913.

JOHN PAUL HEALY, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 6, 1924.

John Paul Healy was born in Washington, D. C., on March 31, 1857. After receiving a public school education, he went into the building industry, studying meanwhile under private tutors.

Mr. Healy followed construction work for several years, later becoming a Superintendent. In 1900, he was appointed an Assistant Inspector of Buildings in the Building Department of the District of Columbia. He specialized in the computation and design of structural steel and, later, in the checking and construction of reinforced concrete.

In 1909, Mr. Healy became the Principal Assistant Inspector of Buildings and, in 1918, he was appointed Inspector of Buildings in direct charge of the Building Department of the District of Columbia, which position he held until his death. His long experience, training, thoroughness, and quiet conscientious character particularly fitted him for such work, and he became a recognized expert.

His untimely death is mourned not only by his wife and eight children, but also by his associates, who recognized in him a man who through his own industry and sterling character had won the admiration and confidence both of his colleagues and of the public whom he so ably served.

Mr. Healy was elected an Associate Member of the American Society of Civil Engineers on December 6, 1910.

* Memoir prepared by Morris Hacker, Esq., Washington, D. C.

THOMAS TYRRELL HEIGES, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 10, 1924.

Thomas Tyrrell Heiges was born at York, Pa., on September 19, 1871, the son of Dr. Jacob D. Heiges and Anna (Smith) Heiges. He attended the public schools of York, and spent three years at the University of Pennsylvania, Philadelphia, Pa.

In 1894 and 1895, Mr. Heiges served as Rodman, Leveler, and Transitman on surveys for the Susquehanna Power Company, at Conewingo, Md. Beginning in the fall of 1895, he worked for the Wrought Iron Bridge Company, Canton, Ohio, as Draftsman and, later, was Engineer on the construction of the overhead bridge at College Avenue, York, as well as on the preliminary surveys and plans for a bridge across the Susquehanna River, at York Furnace, Pa., and on numerous smaller highway bridges.

In 1898, he was employed by the Norway Iron and Steel Company to design and construct its plant at York. From 1900 to 1904, he served as City Draftsman of York and afterward became Superintendent of Construction for George W. Ruch and Company, on the building of a part of the sewerage system of the city. Later, he acted as Superintendent for the Board of Public Works during the completion of that work.

From 1908 until his death, Mr. Heiges was engaged in private practice at York, during which time he specialized in the design and construction of sewerage systems and improved highways, besides carrying on a general engineering practice.

During the World War he had charge of the erection of the plant and buildings for the York Safe and Lock Company and, at the time of his death, was engaged in planning enlargements and in building a new plant for the York Manufacturing Company.

Mr. Heiges was identified with most of the municipal and major private improvements of the City of York for about twenty years. He served two terms as County Surveyor, and was also connected with the State Highway Department of Pennsylvania.

Mr. Heiges was always a lover of high-class music, and, as a boy, sang in the choir of St. John's Protestant Episcopal Church, at York, of which he was a member.

He is survived by his wife, Mrs. Grace (Goodling) Heiges, and by several brothers.

Mr. Heiges was elected an Associate Member of the American Society of Civil Engineers on February 5, 1908.

* Memoir prepared by J. K. Giesey, M. Am. Soc. C. E.

WILLIAM GEORGE JUENGST, Assoc. M. Am. Soc. C. E.*

DIED MARCH 5, 1924.

William George Juengst was born on April 6, 1884, on a farm about four miles from North Vernon, Ind. He was the eldest son of Charles F. and Auguste (Schuster) Juengst, both natives of Germany. He attended the Grammar School near his father's farm and was graduated from the High School at North Vernon.

From September, 1904, to June, 1905, Mr. Juengst was engaged under H. F. Juengst and C. J. Eld, Jr., Members, Am. Soc. C. E., in East St. Louis, Ill., on the construction of a 4 000 000-gal. settling basin, extension of intakes, and other work. He then attended Purdue University, Lafayette, Ind., from 1905 to 1909 and was graduated as Bachelor of Science in Mechanical Engineering. During his vacations from June to September in 1905 and 1906, he worked on the construction of the purification plant and pumping station of the South Pittsburgh Water Company, under John N. Chester, M. Am. Soc. C. E., and Mr. Eld. In 1908 and 1909, he was employed as Draftsman by the American Water Works and Guarantee Company, Pittsburgh, Pa., under G. W. Biggs, Jr., M. Am. Soc. C. E.

In August, 1909, Mr. Juengst was sent to Birmingham, Ala., and acted as Assistant Engineer on the construction of Lake Purdy Dam, Rosedale Pump Station, and the improvement of the North Birmingham Pump Station, under Messrs. Juengst and Eld, until 1911. He was then made Engineer in charge of construction at the different plants of the American Water Works and Electric Company, at New Castle, Pa., Wichita, Kans., Joplin, Mo., St. Joseph, Mo., Chattanooga, Tenn., Birmingham, Ala., and Keokuk, Iowa. His work in these plants consisted of reconstructing pump stations and building filters, laying pipes, moving and erecting machinery, etc.

From 1913 to 1916, Mr. Juengst was Superintendent of the Keokuk Water-Works Company, at Keokuk, Iowa. He was then sent to East St. Louis, Ill., as Engineer in charge of the construction of an 8 000 000-gal., concrete filter plant, a 3 000 000-gal., covered, clear-water and a 5 000 000-gal., sedimentation basin, and a modern coagulating plant.

After the completion of this work, Mr. Juengst had charge of construction work in Muncie, Ind., Wichita, Kans., and since 1921, at Huntington, W. Va., where he rebuilt the entire pumping station, and constructed a sedimentation basin and concrete filter plant. His last work was the construction of a new low-service station on the Ohio River a mile above the mouth of the Guyan River. The work consisted of the construction of a pump pit 75 ft. deep, laying two intake pipes into the Ohio River and a force main across the Guyan River to the sedimentation basin at the high-service station, and the erection of two vertical motor-driven centrifugal pumps. This work was entirely finished when Mr. Juengst was taken ill with appendicitis, which resulted in his death.

* Memoir prepared by George W. Biggs, Jr., M. Am. Soc. C. E.

Mr. Juengst was married on December 24, 1911, to Effie Jane Gray of North Vernon, Ind., who died at Oakdale, Iowa, in 1916. Of this union, there were three children, two girls and a boy; the latter died in 1917. He was married again in East St. Louis, Ill., in July, 1920, to Myrtle B. Lively, who, with his two daughters, survives him.

Of sterling character, Mr. Juengst was possessed of a quiet, pleasant disposition. He was a hard, industrious, conscientious worker and devoted to his family and parents.

He was especially fitted for construction work of large and difficult character. He was a good organizer, and was admired and respected by all who worked under him and were associated with him, as well as by all his superior officers.

One of the Faculty of Purdue University, Professor L. V. Ludy, of the Department of Experimental Engineering, writes of Mr. Juengst, as follows:

"I knew him quite well as he was a student in my classes for two years. He was a thorough student, maintained a high standing in his classes, was prominent in student activities, and was very popular with his classmates. The 1909 *Purdue Débris,* the Senior Year Book, had the following to say about him: 'Although quiet and unassuming in manner, he is a keen student of human nature with an insight into the affairs of men that marks him as a born leader. Resourcefulness and manly dignity have always been his distinctive characteristics.' "

Mr. Juengst was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

SWAN AUGUST KALBERG, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 5, 1922.

Swan August Kalberg was born at Orsos, Sweden, on September 20, 1886. He came to the United States in his early childhood and was educated in the public schools of New Britain, Conn., having been graduated from the High School in 1904. Later, he entered Cornell University from which he was graduated in 1910 with the degree of Civil Engineer.

Mr. Kalberg's first practical engineering experience was with the American Bridge Company and the Berlin Construction Company. He resumed his studies at Cornell University in the fall of 1908, returning to his position with the Berlin Construction Company in June, 1909, where he remained until October. After his graduation, he was employed by this Company from July, 1910, to March, 1911.

In March 1911, he served for one year as Structural Designer for Stone and Webster Engineering Corporation at the main office in Boston, Mass., and was then transferred to Keokuk, Iowa, in connection with the construction of the hydro-electric power station at the Keokuk Dam. After his return to Boston Mr. Kalberg was engaged on the design of reinforced concrete and steel work for the same firm.

In 1914, he accepted the position of Reinforced Concrete Designer with the Aberthaw Construction Company and remained with that firm until July, 1915. He then entered the employ of the F. T. Ley Company at Springfield, Mass., as Structural Engineer in charge of design, his work including the design of many large undertakings of the Company during the World War. The illness from which he never recovered came at the end of this strenuous period and resulted in his death on November 5, 1922.

Mr. Kalberg was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

* Memoir prepared by S. M. Kalberg, Esq., Boston, Mass.

GEORGE ALLEN KYLE, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 17, 1924.

George Allen Kyle, the son of Z. D. and Comfort O. Kyle, was born at Tobasco, near Cincinnati, Ohio, on September 21, 1857.

Mr. Kyle began his professional career in July, 1876, and from that time until 1895, was engaged in railroad work, as follows: With the Cincinnati and Eastern Railway Company, as Chainman and Instrumentman on surveys and construction; the Chicago, Belleville, Cascade and Western Railroad Company, as Levelman on surveys and construction; the Chicago, Milwaukee and St. Paul Railway Company, as Instrumentman on the Sabula Bridge, over the Mississippi River, between Savanna, Ill., and Sabula, Iowa, and Transitman on surveys, and Resident Engineer in charge of the construction of two 15-mile sections of the road; the Cincinnati and Eastern Railway Company, as Locating Engineer and Resident Engineer on the construction of 20 miles of that line in Ohio; the Dubuque and Northwestern Railway Company as Locating Engineer; the St. Paul and Northern Pacific Railroad Company, as Topographer; the Chicago and Evanston Railway Company as Instrumentman; the St. Paul and Northern Pacific Railroad Company, as Resident Engineer in charge of 5 miles of the road; the Dakota, Duluth, Watertown and Pacific Railroad Company as Locating Engineer; the Chicago, St. Paul and Kansas City Railroad Company as Locating Engineer in charge of 40 miles of that line; the Union Pacific Railroad Company, as Locating Engineer in Wyoming, Nevada, and California; and the Northern Pacific Railroad Company, as Locating Engineer, Resident Engineer, and Assistant Engineer in charge of the construction of part of the Grays Harbor Branch in Washington and the Wallace Branch in Idaho.

In 1895, Mr. Kyle went to South Africa with Mr. John R. Stephens, as Engineer in charge of tunnels on the Simmer and Jack Gold Mining Properties in the Transvaal. He remained in this position until the Jamieson Raid (in which he took part) when he was forced to leave the country. He then returned to the United States and to the employ of the Northern Pacific Railway Company. In 1897, he held the position of Assistant Engineer in charge of the construction of the Belmont Branch in Manitoba, Canada, and, in 1898, was in charge of surveys in Southeastern Washington. From December, 1898, to December, 1900, he had charge of surveys for and the construction of the Palmer Cut-Off, an important main-line change near Seattle, Wash., and shortly afterward was made Division Engineer of the Western Division, in which capacity he served until 1902.

At this time the Grand Trunk Pacific Railway was started with Mr. Stephens as Chief Engineer and Mr. Kyle went with that Company as Principal Assistant Engineer. He remained in this position until 1904 when the agitation against the employment of American engineers on work financed by the Canadian Government forced him to resign and he returned to the United States. He was employed on special work for the Northern Pacific Railway

* Memoir prepared by R. F. Heckman, Esq., San Francisco, Calif.

Company for a few months and then went to Alaska as Engineer of Surveys for the Alaska Central Railway. Surveys were made from Seward to Fairbanks, and about 40 miles of line from Seward to Anchorage was built and is now a part of the Government Railway. Government interference and the failure of a Canadian bank largely interested in financing this project forced the stoppage of all work, and Mr. Kyle returned to Seattle where he was made Assistant Chief Engineer of the western part of the Puget Sound Extension of the Chicago, Milwaukee and St. Paul Railway. He remained in this position until the completion of the work, when he was made Chief Engineer of the Oregon Trunk Railway of which John F. Stevens, Hon. M. Am. Soc. C. E., was President. Shortly afterward, Mr. Kyle became Vice-President and General Manager of the Oregon Trunk Railway and the Oregon Electric Railways, which positions he held until 1911 when he resigned to enter private practice in Portland, Ore.

In 1916 Mr. Kyle went to China as Chief Engineer of the Chinese Government Railways in charge of the Siems-Carey Projects. Surveys were made from the Yangste River south to the South China Sea, approximately 1 000 miles, and from the Pekin-Hankow Railway west to Szechuan Province, approximately 1 000 miles. In 1918, while on a reconnaissance trip, he was captured by Chinese bandits, remaining a prisoner for 52 days in very grave danger. This experience had a great deal to do with hastening his death, as he never fully recovered from it. In 1919, all work on the Siems-Carey Projects ceased, and Mr. Kyle returned to the United States.

In 1920, and until 1922, he was connected with the late J. W. Kendrick, M. Am. Soc. C. E., on a proposed railway in Brazil. From 1923 to his death on November 17, 1924, he was engaged as a Consulting Engineer in private practice on the Pacific Coast with an office in San Francisco, Calif. His home in Berkeley, Calif., and all his effects were destroyed in the fire of 1923.

Mr. Kyle's love for outdoors fitted him admirably for the positions he held during his life. He had unusual ability as a mathematician and was possessed of a keen mind, and was without doubt one of the best railway location and construction engineers of his time. This estimate has been confirmed by E. J. Pearson, M. Am. Soc. C. E., who writes as follows:

"He was possessed of the ability, in a remarkable degree, to visualize the relationship of these features, remotely distant, controlling the development of supported grades of unusual length; to associate with each other, within immensely large areas, the opportunities and the restrictions incidental to the topography over an extended range; and through this natural ability, Mr. Kyle's success as a Locating Engineer, was marked. His appreciation of his associates was deep and keen; his loyalty to them unquestioned; and his own abilities were always supplemented by effective co-operation."

On June 18, 1893, he was married to Mildred Corbin, of St. Louis, Mo., who, with two sons, Marion and Hugh, and one daughter, May, survives him.

Mr. Kyle was elected an Associate Member of the American Society of Civil Engineers on February 2, 1892.

ALONZO CHURCH LEE, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 1, 1923.

Alonzo Church Lee, the son of Alonzo Church and Sallie Whitmore Lee, was born in Columbus, Miss., on July 16, 1888. On his mother's side, Mr. Lee was of English-Virginia ancestry, related to the Epes family and to Thomas Jefferson. On his father's side he was descended from the South Carolina branch of the Lee family, his father being a double cousin of Gen. Stephen D. Lee. One of his paternal great-grandfathers, Dr. Alonzo Church, was a noted educator of the South, who for thirty years was President of the University of Georgia. Mr. Lee was proud of his ancestry and felt that in it he had a heritage which was at once an obligation and an inspiration.

Church Lee, as he was called by all who knew him well, received his early education in the public schools of Mississippi. Later, he attended Mississippi College, a preparatory school at Clinton, Miss. He studied for two years at Tulane University, New Orleans, La., and after four years of work at the University of Mississippi, received in 1911 from that institution the degree of Bachelor of Engineering in Civil Engineering. Five years later, on the presentation of a thesis on "Muscle Shoals", his Alma Mater bestowed on him the degree of Civil Engineer.

Mr. Lee's first experience in engineering work was during 1907-08—the year that elapsed between his leaving Tulane and entering Mississippi University. This year was divided between inspection of lock and dam construction on Ouachita River under the United States Engineer Department and general office and field work under the City Engineer of Jackson, Miss. Thereafter, he worked during vacation periods with the City Engineers of Jackson and Oxford, Miss., and with the Illinois Central Railroad Company. In his Junior year, he was a Student Assistant Instructor in Surveying and Drafting.

On graduation, Mr. Lee again served as Inspector on the Ouachita River until the flood period stopped work when he was employed by the Tennessee Coal, Iron and Railroad Company in connection with construction of a modern mine surface plant at Edgewater, Ala. From July to October, 1912, he was Concrete Inspector on tunnel and power-house construction at Tallulah Falls, Ga. He then spent three months, until the work was completed, laying out and supervising a concrete pile foundation for a seventeen-story office building in Atlanta, Ga. Afterward, Mr. Lee was engaged for two months with the Sloss-Sheffield Steel and Iron Company on mine plant construction when he resigned to accept a position with the Engineer Corps of the U. S. Army, as Chief of Party on the Cumberland River, first on a precise level survey and, later, on a topographic survey and lock and dam location, in preparation for the extension of the canalization of the river. In April, 1915, he was transferred to work on the Tennessee River where he assisted in the field and on preliminary plans for the development of power and improvement of navigation of the Muscle Shoals District.

* Memoir prepared by W. S. Winn, Assoc. M. Am. Soc. C. E.

On the completion of the Muscle Shoals investigations, Mr. Lee decided to enter a branch of his profession which would provide a more permanent employment, and accordingly took the Sales Course at four of the plants of the Ingersoll-Rand Company. In December, 1916, he became connected as Sales Engineer with the Company's office in Birmingham, Ala.

When the United States became involved in the World War, Mr. Lee at once responded to the call of the blood of his warrior ancestors, and in August, 1917, was admitted to the training camp at Fort Oglethorpe, Georgia. On November 27, 1917, he received a commission as First Lieutenant of Infantry and was assigned to the 17th Machine Gun Battalion. His principal stations in this country were Chickamauga Park, Georgia; Camp Wadsworth, South Carolina; Camp Mills, New York; and Camp Dix, New Jersey. In July, 1918, he went overseas, where he took part in the Meuse-Argonne drive. Returning to the United States a year and ten days later Mr. Lee was honorably discharged from the Army on August 6, 1919, and immediately returned to the service of the Ingersoll-Rand Company, being attached to its St. Louis office. After three years, however, he resigned on account of ill health and went to Southern California where he engaged in the real estate business, hoping that a change of work and climate would restore his health.

On September 1, 1923, he went for a swim with a friend at the Santa Monica, Calif., bathing beach. Mr. Lee was in his usual good spirits. When the time came to leave the beach, the two who had been sunning in the sand raced to the ocean for a final dip. The friend came out, but Mr. Lee was never seen alive again and no one knows just how he was drowned. His body, recovered a few days later, was cremated as he had requested it should be, and his ashes were scattered with white roses on the ocean to which he had forfeited his life.

Mr. Lee was well equipped mentally, morally, and physically, to meet the struggles of life. Descended from educators and college-bred ancestors on both sides of his family, he early determined to have a college education and secured it largely by his own efforts. In his youth he joined the Presbyterian Church and remained a consistent member until his death. He was a Knight Templar, and a member of the American Legion and of the Kappa Alpha Fraternity.

He was a clean-cut athlete and loved all kinds of sports. He was recognized as one of the most brilliant athletes the University of Mississippi ever produced, starring as Captain of the basket-ball team and as Fullback on the "Ole Miss" football team of 1910. He carried a full measure of enthusiasm into whatever he was doing—whether work or play. With all his strength and fighting vim, his heart was as tender as that of a child. His devotion to and tender care of his mother was beautiful to see. He was unmarried and is survived by his mother and a younger brother, Frank W. Lee, of Tulsa, Okla.

Mr. Lee was elected an Associate Member of the American Society of Civil Engineers on June 23, 1916.

BERTRAND CLIFFORD MARTIN, Assoc. M. Am. Soc. C. E.*

DIED JUNE 8, 1924.

Bertrand Clifford Martin, the son of Nelson H. and Estelle G. Martin, was born at Fort Fairfield, Me., on November 18, 1879. He was graduated from the University of Maine, at Orono, in 1901, with the degree of Bachelor of Civil Engineering, his father having been graduated from the same college. During his college days, Mr. Martin was a fine athlete and for two years was a pitcher on the University baseball team.

In September, 1901, he joined the staff of the Bangor and Aroostook Railroad Company, as Levelman on the survey of the Fish River Railroad. He remained with this Company until 1903, serving at different times as Transitman and Assistant Engineer in charge of seven miles of railroad construction at Portage, Me., and of a four-mile branch line of the Bangor and Aroostook Railroad at Van Buren, Me., which included the construction of an 800-ft. timber trestle on a pile foundation.

From June, 1903, until November, 1906, he was employed by Westinghouse, Church, Kerr, and Company, of New York, N. Y., first as Instrumentman on railroad location in the western part of Virginia, and, subsequently, on the location and construction of foundations for steel poles of the high-tension line on the electrification of the Long Island Railroad.

On the completion of the project in July, 1905, Mr. Martin was sent to Ohio where he was employed on railroad work, in charge of a party, locating twenty miles of the double-track interurban electric line between Vanport, Pa., and Wellsville, Ohio. He returned to the New York Office of the Company in March, 1906, and drew plans of track layouts for the railroad terminal at Toronto, Ont., Canada, and checked the track layout for the Pennsylvania Terminal in New York.

Mr. Martin accepted the position of Assistant Engineer in the office of the Designing Engineer with the New York Central and Hudson River Railroad Company, in November, 1906, where he was engaged in designing and estimating the cost of track work, passenger and freight yard layouts, masonry, water tanks, structures, etc. He displayed such marked ability in this position that he was placed in charge of the Resident Office, at Watertown, N. Y., in 1909, afterward being transferred to Utica, N. Y., in a similar capacity.

From 1913 to 1916, Mr. Martin was Engineer in charge of the four-tracking of the Hudson Division of the New York Central and Hudson River Railroad Company, with headquarters at Poughkeepsie, N. Y., and from 1916 to 1920, he handled the preliminary surveys, location, etc., on the Hudson River Connecting Railroad, leaving the work in excellent condition for construction. He was also in charge of the Mohawk and St. Lawrence Divisions of that railroad.

* Memoir compiled from information on file at the Headquarters of the Society.

After the World War, when the Government faced the task of returning the railroads to their owners, Mr. Martin was transferred to the New York Office of the Company where he was placed in charge of a re-valuation of the railroad properties. On the completion of this work, he returned to his position as Resident Engineer on the Hudson River Connecting Railroad with headquarters at the Divisional Office at Albany, N. Y.

Owing to his familiarity with construction details, and following a lapse in field activities, he was placed in charge of roadway completion reports, continuing in this position until the authorization of the construction of the Hudson River Connecting Railroad in 1922. Mr. Martin was then made Resident Engineer in charge of the Castleton Bridge, spanning the Hudson River and carrying the Hudson River Connecting Railroad into Selkirk Yard, including connections with the Boston and Albany Railroad and the Hudson Division of the New York Central Railroad, and held this position at the time of his death which occurred on June 8, 1924, as a result of pneumonia that developed after a short illness. He is survived by his wife, Mrs. Adelaide W. Martin, two sons, George Nelson and Charles Bertrand Martin, his mother, and a brother, Charles H. Martin.

Mr. Martin was competent and faithful in his work and always gave of his best to his employers by whom he was held in high esteem. He was of a cheerful disposition, interested chiefly in outdoor sports, his home, and his work.

He was a Mason, a member of the American Railway Engineering Association, and of the Columbia County Country Club, where he found recreation in golf and tennis. He was also a member of the Dutch Reformed Church.

Mr. Martin was elected an Associate Member of the American Society of Civil Engineers on May 4, 1909.

EDWARD GILBERT MATHEWS, Assoc. M. Am. Soc. C. E.*

DIED JULY 19, 1924.

Edward Gilbert Mathews, the son of Tertullus G. and Sarah A. Mathews, was born in Brooklyn, N. Y., on December 30, 1883. He attended Adelphi Academy, in Brooklyn, later going to Riverview Military Academy, at Poughkeepsie, N. Y., where he spent three years in preparation for Princeton University. He entered Princeton in the fall of 1903, and was graduated in 1907 with the degree of Civil Engineer.

Mr. Mathews' first position was with the Hennebique Construction Company, on the design of reinforced concrete structures. In 1909, he entered the employ of the United Fireproofing Company, Engineers and General Contractors, specializing in reinforced concrete construction. Later, he became Vice-President of this Company, which position he held until 1913, when he was elected President. In 1918, Mr. Mathews was retained by the Taintor Company as Consulting Engineer in connection with certain additions to the manufacturing plant. In 1919, he was retained permanently by the Taintor Company, at the same time being Vice-President and Director of the United Fireproofing Company. On February 4, 1920, he was elected a Director of the Taintor Company.

In January, 1924, he resigned as Vice-President of the United Fireproofing Company, to accept the position of Treasurer of the firm, still remaining actively engaged with the Taintor Company.

He was held in high esteem by his many friends and business acquaintances. He was a very able engineer and organizer, showing promise of reaching an even higher position in his profession. He also took a great interest in the activities of the communities in which he resided.

He was a member of the Princeton Engineering Association, the Princeton Club of New York, and the Princeton Elm Club.

Mr. Mathews and Miss Estelle Smith were married on April 20, 1908, in Brooklyn, where they made their home. Mr. Mathews' death occurred at his summer home in Hartsdale, N. Y. Besides his wife and mother, he is survived by two children, Frances and Gilbert.

Mr. Mathews was elected an Associate Member of the American Society of Civil Engineers on June 24, 1914.

* Memoir prepared by Messrs. James Dusenbery and Lemuel Osborne, New York, N. Y.

HUGH LESTER MAYERS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 25, 1924.

Hugh Lester Mayers was born in Kennedy, Ala., on May 8, 1892. He attended the Vernon High School in Lamar County and was graduated from the Alabama Polytechnic Institute in 1914, with the degree of Bachelor of Science in Civil Engineering.

Following his graduation, Mr. Mayers served as Rodman and Instrumentman with the Alabama Power Company from August, 1914, to August, 1915, and as Rodman and Assistant with the Southern Railway Company, from August, 1915, to December, 1916. In January, 1917, Mr. Mayers entered the Chicago Office of the Illinois Central Railroad Company as Draftsman where he remained until the United States entered the World War. Receiving a leave of absence, he returned to Alabama and enlisted as a Private in the U. S. Army. Two months after his enlistment, he was sent overseas with Company B, 306th Engineers, 81st Division, and soon thereafter was commissioned a Second Lieutenant. After being honorably discharged Mr. Mayers returned to Chicago and resumed his work with the Illinois Central Railroad Company.

In April, 1920, he was appointed to the position of Resident Engineer in charge of bridge construction near Montgomery, Ala., by the State Highway Commission of Alabama. He was one of the most efficient engineers connected with the State Highway Department, always loyal to the State and conscientious in the performance of duty. In July, 1923, he resigned this position to accept a more lucrative one as Consulting Engineer with the Stanley and Singer Construction Company, Contractors, of Lafayette, Ala., in charge of all engineering in connection with Federal Aid bridge and road work, which position he held at the time of his death.

On Saturday morning, October 25, 1924, Mr. Mayers and Mr. T. E. Stanley were inspecting a proposed State road project in Russell County, Ala., and while crossing the Seaboard Air Line Railway tracks near Hurtsboro, in an automobile, were struck by a train and instantly killed. Both men were very popular in the State, Mr. Stanley being one of the most prominent road contractors in Alabama.

Mr. Mayers was highly esteemed by all who knew him and his scores of friends were greatly shocked to learn of his untimely death.

The State Highway Commission in session on November 12, 1924, passed the following resolutions:

"*Whereas,* T. E. Stanley as Contractor and Hugh L. Mayers as Engineer, have been connected with the State Highway Department for a number of years, Mr. Stanley being a contractor of high standing and integrity and Mr. Mayers being directly with this Department for a number of years, and later Engineer for the firm of Stanley and Singer and at all times known for his high standing as a gentleman and engineer of ability and rugged honesty; and

* Memoir prepared by W. S. Keller, M. Am. Soc. C. E.

"Whereas, The State Highway Commission has learned with profound sorrow of the untimely death of these two excellent men on October 25, 1924, at Hurtsboro, Alabama,

"Now, Therefore, Be It Resolved, that the State Highway Commission in session on this the 12th day of November, 1924, goes on record as greatly deploring the loss of Messrs. Stanley and Mayers, their passing being an irreparable loss to their families and a misfortune to the State Highway Department.

"Be It Further Resolved, that our deepest sympathy be expressed to the bereaved families of our departed friends, and that a copy of these resolutions be spread on the minutes of the Commission."

Mr. Mayers was elected an Associate Member of the American Society of Civil Engineers on February 25, 1924.

EMIL ROBERT OLBRICH, Assoc. M. Am. Soc. C. E.*

DIED MAY 25, 1924.

Emil Robert Olbrich, the son of Florian and Mary Olbrich, was born on May 22, 1891, in Philadelphia, Pa., where he received his early education. He was graduated from the Philadelphia-Rutledge Grammar School in 1905, and from the North East Manual Training High School, in Philadelphia, in 1908.

Mr. Olbrich was associated with the *Philadelphia Public Ledger* as Advertising Soliciting Agent for a short time, and then entered Pennsylvania State College, from which he was graduated in 1915 with the degree of Bachelor of Science in Civil Engineering.

He then joined the staff of the Bureau of Highways, Philadelphia, where he served for about two years as Highway Engineer-Inspector, on the construction and maintenance of all types of city pavements and urban roads, interspersed with laboratory testing of paving materials.

In April, 1917, Mr. Olbrich accepted a position with the Barrett Company, in charge of a contract with the Pennsylvania State Highway Department for the surface treatment of 1 000 miles of water-bound macadam in Western Pennsylvania. In November of the same year he was appointed Assistant Professor of Civil Engineering, at the Agricultural and Mechanical College of Oklahoma, serving in this capacity until 1920, except for a period of six months in 1918-19 when he obtained leave of absence and was detailed as Instructor in the Naval Officers' Training School, U. S. Navy.

Mr. Olbrich then entered the University of Michigan, and was graduated therefrom in June, 1921, with the degree of Master of Science in Highway Engineering. He served the remainder of that year with the Dow Chemical Company, of Midland, Mich., and in January, 1922, was appointed Construction Engineer with the North Carolina State Highway Commission. He held this position with distinction until October, 1923, when he resigned to serve as Assistant to the Director of the Advisory Board on Highway Research, National Research Council, with headquarters at Washington, D. C., where he remained until his death.

Mr. Olbrich was only thirty-three years of age at the time of his death, and was advancing rapidly toward a distinguished career along the lines of scientific research in highway engineering, for which work he displayed marked aptitude. He rendered particularly valuable service to the North Carolina State Highway Commission, by his assistance in the study and development of the sand asphalt pavement, many miles of which are now being successfully constructed by that Commission. He was widely known among members of the Engineering Profession, to whom the knowledge of his death came as a great shock. His loss is deeply mourned.

He was unmarried, and is survived by his father and mother, and by two brothers and a sister.

Mr. Olbrich was elected an Associate Member of the American Society of Civil Engineers, on November 9, 1920.

* Memoir prepared by Charles M. Upham, M. Am. Soc. C. E.

CLARK OLDS, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 16, 1922.

The death on August 16, 1922, of Clark Olds, Assoc. M. Am. Soc. C. E., brought to an end a kindly, honorable, and useful life. Starting as an engineer, he changed to the profession of law, became prominent in civic affairs, and left a record of conspicuous service to his community in furthering engineering works. His career illustrates the influence and value of an engineering training.

Clark Olds, the son of Lewis Wilson and Louisa Everett (Ackerly) Olds, was born near Erie, Pa., on July 14, 1850. His family removed from New England to Pennsylvania in pioneer days. After attending the public schools and Erie Academy, he entered the University of Michigan and was graduated therefrom as Bachelor of Science in 1870. He received his Master's Degree two years later.

During vacations and after his graduation, Mr. Olds was engaged with the United States Lake Survey. In 1872 he gave up this work and turned his attention to the study of law. After two years at the University of Leipsic in the study of Roman and international law, he returned to Erie, entered the law office of William Benson, Esq., and was admitted in 1876 to the Erie County Bar and subsequently to practice before State and Federal Courts. Thereafter, for nearly fifty years, Mr. Olds pursued a successful general practice, specializing in Admiralty law.

He was married on December 13, 1876, to Livia Elizabeth Keator, of Cortland, N. Y. Of their three children, one survives, Irving Sands Olds, a graduate of Yale University and a successful attorney of New York City.

Mr. Olds was prominent in the development of the City of Erie, its waterworks, harbor, docks, and public parks and institutions, and the activities of its Chamber of Commerce. During his régime as Water Commissioner and President of the Board, the intake was extended, the pumping and distribution systems were reconstructed and enlarged, and a generous supply of good water was provided for the growing needs of the municipality with intelligence and foresight. The improvement became a factor in the location of manufacturing interests at Erie. As Chairman of the Harbor Commission, his efforts were largely responsible for securing the legislation and funds that resulted in much needed water-front improvements. Construction in this connection was under the active charge of his Commission and, on its completion, a substantial unexpended balance was returned to the State. Always interested in the development of the harbor and peninsula, his constant work in that direction has met with success and is expected to be followed by the extension of the Park System on a large scale. He also took active part in the raising of Perry's flagship, the *Niagara,* for the Centenary of the Battle of Lake Erie.

Mr. Olds filled many posts of honor and trust at various times. He was President of the Erie Chamber of Commerce and of the Erie County Mutual

* Memoir prepared by Frederick C. Noble, M. Am. Soc. C. E.

Fire Insurance Company and, at the time of his death, was a Director of the Erie Trust Company and of the L. W. Olds Real Estate Company and President of the Erie and Lakeside Cemeteries. For many years, he was a delegate to the International Peace Conference at Lake Mohonk, N. Y. During the World War, he served as Chairman of the Draft Questionnaire Board and on the local Liberty Loan Committee. His last public service was as Chairman of the Committee that effected a revision of the rules of practice of the Orphans' Court. Although he had never held an elective office, he was active in politics at campaign times, and in 1904 was a delegate to the Chicago Convention that nominated Theodore Roosevelt for President.

Besides being an able lawyer and a public-spirited citizen, Clark Olds was an honest, fearless, upright man of the highest ideals. His whole-hearted, sympathetic nature readily responded to every worthy undertaking, and his sterling qualities, with the advantage of a pleasing personality, made and held a host of friends. The late Alfred Noble, Past-President, Am. Soc. C. E., his classmate and associate on the Lake Survey, reciprocated a firm friendship with Mr. Olds and held him in lifelong esteem.

Mr. Olds was elected an Associate Member of the American Society of Civil Engineers on March 1, 1899.

ARTHUR DICKSON PRINCE, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 21, 1925.

Arthur Dickson Prince, the son of Christopher Prince, Jr., and Julia Dickson Prince, was born at Glen Cove, Long Island, N. Y., on October 21, 1870. He was the twelfth in the direct line of descendants to inherit the estate on which he was born and where his death occurred, this property having been purchased by his ancestors directly from the Indians.

At the age of nineteen, he entered the old School of Mines of Columbia College, now Columbia University, and was graduated with his class in 1893 with the degree of Civil Engineer. While at Columbia he took an active and prominent part in college activities, rowing on his Freshman and the three succeeding "Varsity" crews of the College. He continued his activity in sports, being an ardent yachtsman, competing in many races on Long Island Sound and winning many prizes.

Following his graduation, Mr. Prince was employed as Superintendent of the George F. Swift Construction Company on contract work in New York, N. Y., as Draftsman for Professor H. S. Munroe, of Columbia College, on coal mining work, and from March, 1894, to March, 1896, by the Metropolitan Traction Company on the construction of the Lexington Avenue Cable Road of New York, in responsible charge of a part of the construction of this road. From the spring of 1896 until the fall of 1900, he was in the employ of various Departments of the City of New York in the capacity of Assistant Engineer in charge of construction.

On the formation of the Engineering Department of the Board of Rapid Transit Commissioners, organized for the construction of New York's first subway, Mr. Prince was transferred to that organization as Assistant Engineer on October 1, 1900, and was assigned to the First Division. He continued his connection with this Division until the time of his death, having served for the past fifteen years as Senior Assistant Division Engineer.

In 1905, previous to the construction of the tunnel of the Pennsylvania Railroad under the 33d Street Station of the Fourth Avenue Subway, Mr. Prince was designated to conduct investigations relative to recording the vibrations caused by the blasting to be done in tunneling beneath the subway. He designed and constructed an instrument on the principle of the vertical pendulum seismograph for recording amplitude and period of vibrations in three directions. This instrument was operated during the period of blasting operations in the Pennsylvania Tunnels in the vicinity of the Fourth Avenue Subway and recorded successfully the vibrations caused by this blasting. A notable occurrence in the operation of the instrument was the recording of the major earthquake shock which destroyed the City of San Francisco, Calif., in April, 1906, the violence of the earth movement having been sufficient to swing the recording needle outside the edge of the record sheet. This instrument was later operated in the Times Building for recording vibrations due

* Memoir prepared by George Hallett Clark, M. Am. Soc. C. E.

to the operation of subway trains and in several other buildings adjacent to subways. At present, it is on loan to the United States War Department and is being used in the Astoria Gas Tunnel in connection with blasting in the East River.

In a service of twenty-five years with the City of New York in the development of its rapid transit facilities, Mr. Prince has typified the trained and educated Engineer whose ability, constantly increasing knowledge of special conditions, and devotion to duty have made possible the prosecution of great public works to successful completion, notwithstanding the inevitable changes in the directing personnel of City and State Commissions.

On June 7, 1900, Mr. Prince was married to Emily Jackson, who, with a son, James D. Prince, survives him.

Mr. Prince was elected a Junior of the American Society of Civil Engineers on May 5, 1896, and an Associate Member on October 3, 1900.

LEROY TALLMAN Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 30, 1924.

Leroy Tallman was born at Portsmouth, R. I., on April 30, 1873. He obtained his elementary education in his native town, and was graduated from the Fall River High School, Fall River, Mass.

From June, 1893, to September, 1897, Mr. Tallman served as Rodman, Instrumentman, and Chief of Party with Wolstenholme and Buffinton, Civil Engineers, of Fall River. From October, 1898, to April, 1900, he was with E. I. Marvell, Civil and Mill Engineer, as Engineer in charge of the office at Fall River.

From April, 1900, to September, 1904, Mr. Tallman was engaged as Assistant Engineer in charge of the design and construction of reservoirs and a filter system for the Newport Water Company, Newport, R. I.

In April, 1905, he joined the forces of the O'Rourke Engineering Construction Company as Superintendent of Construction on the Pennsylvania Railroad Tunnels under the Hudson River, from 33d Street, New York, N. Y. On the completion of the tunnels, he was transferred to work on the caisson foundations of the Hudson Terminal Buildings, acting as Assistant Engineer. He was then given charge of the building of the important project known as the Baltimore Sewerage System, as Superintendent for M. A. Talbot Company, and working under the direction of Mr. Calvin Hendrick.

Mr. Tallman took charge thereafter of the construction of the Degraw Street Tunnel, connecting Gowanus Canal with Gowanus Bay, New York, representing the John Pierce Company. On the completion of this work, he joined the organization of Snare and Triest, and built the Long Island City approaches to the Queensborough Bridge. Later, he directed harbor and seawall work at Michigan City, Ill.

On his return to the East in the spring of 1910, Mr. Tallman became connected with the Pittsburgh Contracting Company, as Superintendent on Contract No. 52 on the Elmsford Aqueduct, Elmsford, N. Y. This Company was a subsidiary of Booth and Flinn, Limited, with which firm Mr. Tallman continued until his death. He had been, therefore, a member of the Booth and Flinn organization for fourteen years, during which time he was in charge of the following construction projects: Several sections of the Passaic Valley Sewerage System, most of which was through soft ground and under compressed air; the Clark Street Tunnel for the Transit Commission, City of New York; several sections of sewers for the Borough of Brooklyn, City of New York; a number of important tunnel sewer contracts for the Cities of Detroit, Mich., and Toledo, Ohio; and, at the time of his death, Contract No. 4 of the Hudson River Vehicular Tunnels (the under-river portion on the New Jersey side).

* Memoir prepared by S. M. Rutledge, Esq., New York, N. Y.

Mr. Tallman is survived by his wife, having no other near relatives. He was a member of Eureka Lodge, A. F. and A. M., and of Portsmouth Royal Arch Chapter, and was President of the Malba Field and Marine Club, Malba, Long Island.

Mr. Tallman was elected an Associate Member of the American Society of Civil Engineers on February 2, 1909.

DANIEL ANDERSON TOMLINSON, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 7, 1924.

Daniel Anderson Tomlinson was born in Detroit, Mich., on April 17, 1890. He received his early education in the Public Schools of Detroit and Waukegan, Ill., and spent one year at Lake Forest College, Lake Forest, Ill. He then entered the Massachusetts Institute of Technology from which he was graduated with honors in 1912. Later, he took special work at Armour Institute, Chicago, Ill.

Mr. Tomlinson began his engineering career as Draftsman on the Subway Division of the Department of Engineering, City of Chicago, and held this position from September, 1908, to July, 1909. His work consisted of the preparation of maps showing the location of utilities in down-town streets; preliminary plans for a proposed down-town subway; traffic studies on surface and elevated lines; estimates of future population; and traffic and other studies for a subway report.

During his college vacations in 1910 and 1911 he served as Foreman of a ditching gang for a curb and gutter contractor at the Great Lakes Naval Training Station, Great Lakes, Ill., and as Draftsman for the Chicago Western and Indiana Railroad Company on preliminary plans and studies for track elevation, clearing yard, and terminal changes.

In June, 1912, Mr. Tomlinson returned to the Chicago Western and Indiana Railroad Company, with which he remained until June, 1917. During this period, he held the position of Assistant Engineer making investigations and reports on track elevation, yard construction, valuation, and terminal facilities. He also assisted B. J. Arnold, M. Am. Soc. C. E., on work in connection with his report covering the terminal situation in Chicago, with special reference to the proposed Union Station Ordinance.

From June to November, 1916, Mr. Tomlinson served as Supply Sergeant with Company A, Engineers, Illinois National Guard, which was mustered into Federal Service for five months' duty at San Antonio, Tex. In June, 1917, Company A was again called for duty during the construction of Camp Grant, Ill., and Mr. Tomlinson, then Second Lieutenant, was assigned as Adjutant to the Constructing Quartermaster, with duties similar to those of an Assistant to the Chief Engineer on civil work.

On August 27, 1917, he entered the Second Officers' Training Camp at Fort Sheridan, Ill., and on November 26, 1917, was commissioned First Lieutenant and stationed at Fortress Monroe, Va., as Instructor in Railroad Artillery and Orientation at the Coast Artillery School. On September 26, 1918, he was commissioned Captain in the Coast Artillery. At this time he prepared several bulletins and lectures, including a revised edition of "Coast Artillery Notes No. 11". He received his discharge from the Army on January 25, 1919.

From January to March, 1919, Captain Tomlinson served as Manager for the Central Zone, Professor, and Special Secretary of the U. S. Employment

* Memoir compiled from information supplied by H. C. Blake, M. Am. Soc. C. E., and from data on file at the Headquarters of the Society.

Service at Chicago. The work of this organization was that of obtaining employment for engineers and other professional men then being discharged from the Army and Navy. It was discontinued when Congress failed to appropriate funds for its support.

From April, 1919, to February, 1920, Captain Tomlinson was Assistant Secretary for the American Association of Engineers in charge of railroad and employment work, and from February, 1920, until December, 1921, he was Assistant Engineer in the Technical and Structural Bureaus of the Portland Cement Association. His work during this period consisted in preparing new technical and promotional booklets on concrete, distributing literature, answering technical inquiries, and managing lecture work. From January, 1922, until his death on August 7, 1924, he held the position of Manager of the Railways Bureau of the Portland Cement Association, and as such was engaged in educational and promotional work on railway uses of concrete. He was the author of a series of educational bulletins on this subject and on factors of good concrete work.

In 1917, Captain Tomlinson was married to Leonora Lucas who survives him. He had always been active in various sports, particularly tennis, and his death occurred from heart failure on the tennis courts at Evanston, Ill.

He was a member of the Western Society of Engineers, the Chicago Engineers Club, the Technology Club of Chicago, of which he was Secretary in 1921, and of various other local associations. He was also a Mason, a member of Banner Blue Lodge No. 924, A. F. and A. M., and of the American Legion, Evanston Post No. 42.

H. C. Blake, M. Am. Soc. C. E., President of the Technology Club of Chicago, states that "the number of Captain Tomlinson's friends was limited only by his acquaintanceship, which stretched from one end of the country to the other". Another tribute from one of his many friends, is as follows: "'Tommy' cannot be forgotten; he has worked with and for all in every condition."

Captain Tomlinson was elected an Associate Member of the American Society of Civil Engineers on May 28, 1923.

RICHARD DANA UPHAM, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1924.

Richard Dana Upham, the son of Dr. J. Baxter and Catherine Bell Upham, was born in the old Dana mansion, Manchester-by-the-Sea, Mass., on August 18, 1866. He was graduated from St. Mark's School and, later, from Harvard College in the Class of 1890.

Soon after his graduation, Mr. Upham was appointed to the position of General Superintendent of the International Pavement Company, New York, N. Y., which position he held until the fall of 1893. During this time he made a thorough study of the manufacture of asphalt for paving and other commercial uses and obtained the extensive knowledge of this material that enabled him to develop improvements in its manufacture and that eventually revolutionized the methods then in use.

Mr. Upham continued his investigations during the next three years, having accepted an appointment with the Barber Asphalt Company, in October, 1893. His work with this Company was interesting in its variety, including the erection of the plants at Long Island City, N. Y., on the Schuylkill River, Philadelphia, Pa., and the enlargement of the plant at Trinidad, British West Indies.

In 1896, he opened an office as Consulting Engineer, specializing, among other things, in water-proofing problems. Under Mr. Upham's supervision the Queen Lane and Roxborough Reservoir, in Philadelphia, was successfully repaired. He was also consulted on and directed the water-proofing of the Rapid Transit Tunnel in New York.

In 1912, he was one of the organizers of The Interocean Oil Company, of which he was Vice-President and General Manager, a position he held until his death.

On August 22, 1895, Mr. Upham was married to Elizabeth Rice, of New York, who, with a son and daughter, survives him. He died on September 29, 1924, and was interred in Mount Auburn Cemetery, Cambridge, Mass.

Mr. Upham was a man of high ideals and absolute integrity, a loyal friend, and devoted to his family. He was a member of the University, Harvard, and Lawyers' Clubs, and the New York and Seawanhaka Yacht Clubs. His favorite sport was sailing his schooner yacht *Sea Fox,* on which he lived during the summer. He always commanded her himself and on his last cruise, immediately before his final illness, achieved the feat of bringing her safely through the hurricane of August 25, 1924, off the New England Coast.

Mr. Upham was elected an Affiliate of the American Society of Civil Engineers on October 6, 1896, and an Associate Member on April 3, 1901.

* Memoir prepared by Chandler Davis, M. Am. Soc. C. E.

GEORGE LAWRENCE WATTERS, Assoc. M. Am. Soc. C. E.*

DIED MAY 18, 1924.

George Lawrence Watters was born at St. Clairesville, Ohio, on October 23, 1885. His paternal ancestors came to America in the early part of the Seventeenth Century and settled in Maryland and Virginia. His maternal ancestors who were French Huguenot and German, originally settled at Old Economy, west of Pittsburgh, Pa., and were members of the Harmony Society.

Mr. Watters' father, Leonidas Hamline Watters, spent the greater part of his life in educational work, having been assisted by his wife who was also a keen student and collaborated with him in his writings. Their children were surrounded by an atmosphere of culture and refinement, particular attention being given to the arts, literature, and science.

George Lawrence Watters received his early education in the public schools of Media, Pa., where his father held the position of Principal of the High School. After leaving the Media High School, he entered Swarthmore College at Swarthmore, Pa., and was graduated in the Class of 1906, with the degree of Bachelor of Arts in Engineering.

Immediately after his graduation, Mr. Watters was employed as Assistant Engineer with the American Pipe and Construction Company of Philadelphia, Pa., on the design and construction of masonry and earth dams, reservoirs, stand-pipes, pipe lines, and filtration plants. As such, he served in many and varied capacities from Designing Draftsman to the actual execution and construction of the work in the field, as Resident Engineer.

In 1911, he resigned his position with the American Pipe and Construction Company and accepted that of Hydraulic Engineer with the Lehigh Valley Railroad Company, in which capacity he had charge of the design, construction, operation, and maintenance of the water supplies and fire protection of the Company, with headquarters at Bethlehem, Pa. During his employment with the Lehigh Valley Railroad Company he instituted many improvements in its water supply and installed a number of pumping and gravity supplies and water-softening plants.

In March, 1919, the position of Hydraulic Engineer having been abolished, Mr. Watters accepted that of Special Engineer of the International Petroleum Company, Limited, of Toronto, Ont., Canada, and made a three months' visit to the oil fields in Peru. The particular problem of the Company was to obtain a sufficient and adequate water supply for its operations, and as a result of Mr. Watters' investigations he was sent to Peru in November, 1919, to install a complete water supply system, including the intake, high and low-lift pumping plants, a filtration plant, and a distribution system, with leveling and supply reservoirs and a stand-pipe. The chief difficulty in this work was to obtain sufficient water to meet the demands of the Oil Company with an available annual rainfall that seldom exceeded 2 in.; paradoxically as it may seem,

* Memoir prepared by J. E. Gibson, M. Am. Soc. C. E.

during his engagement in Peru, he was called on to assist in the solution of a flood problem that occurred, due to an unprecedented rainfall or cloudburst.

Mr. Watters completed the water supply work and returned to the United States early in 1922. In June, 1922, after a vacation on the Pacific Coast, he made a third trip of three months to Peru and a fourth trip in January, 1923, for a period of six weeks. These latter trips were inspection, or consultation, trips with the Resident Engineers of the Oil Company in connection with water supply and flood-protection problems.

In May, 1923, Mr. Watters severed his connection with the International Petroleum Company, and accepted a position with the Western New York Water Company as Designing Engineer. He undertook and completed the design of the 16 000 000-gal. gravity and mechanical concrete filter plant a few weeks before his death.

In 1913, while employed with the Lehigh Valley Railroad Company, Mr Watters was unfortunate enough to break his leg while playing baseball and was confined to the hospital for a period of approximately nine weeks. At that time he learned that he had developed kidney trouble, but as he was very optimistic, and reticent about the condition of his health, his friends and acquaintances were not informed as to his true condition. However, it was not until 1923 that his illness became really serious. He died in Bethlehem, Pa., on May 18, 1924.

To those who were associated with him in the employ of the American Pipe and Construction Company of Philadelphia, Mr. Watters endeared himself by his courtesy and consideration of others and his sterling qualities of character. He was painstaking and accurate in research, design, and construction, and was always ready to lend a helping hand in any task, whether in the drawing-room or in the field, and none ever called on him for assistance who did not receive the best he could give.

Mr. Watters was married on March 3, 1916, to Bertha Louise Hark, of Bethlehem. Two children were born to them, Alice, who died in July, 1917, and Louise.

He was a member of the American Water Works Association, the New England Water Works Association, the Engineers' Club of Philadelphia, and of the George W. Bartram Lodge No. 298 F. and A. M., at Media, Pa. He was also a member of the Presbyterian Church and took an active part in its affairs, serving as a Trustee for a number of years.

Mr. Watters was elected an Associate Member of the American Society of Civil Engineers on April 30, 1912.

GAYLORD D. WEEKS, Assoc. M. Am. Soc. C. E.*

DIED MAY 6, 1924.

Gaylord D. Weeks was born at Mechanicsville, Iowa, on November 18, 1878. He prepared for college in the Denison, Iowa, High School, and entered the University of Iowa in 1894, from which he received the degree of Bachelor of Science in Civil Engineering in 1898.

After leaving college, Mr. Weeks was engaged during 1898 as City Engineer of Denison, Iowa. In 1899 and 1900, he served as Rodman, Instrumentman, and Timber Inspector on construction work on the Chicago Northwestern Railway.

From September, 1900, to June, 1902, he was Instructor in Civil Engineering and Mechanical Drawing in the State University of Iowa. From the fall of 1902 to April, 1905, he served as Office Engineer in the office of the Assistant Chief Engineer of the St. Louis and San Francisco Railroad Company, at Springfield, Mo., in charge of the Drafting Office, in which position he obtained considerable experience in the design of railroad yards and structures.

In April, 1905, Mr. Weeks became a member of the contracting firm of Hedges-Weeks Construction Company, and continued with this firm until his death, having had charge of the construction of many important railroad and other concrete structures in various parts of the Middle West. Mr. Weeks was also a Director of the Jarrett-Richardson Paving Company and the Jarrett Construction Company.

On December 31, 1900, Mr. Weeks was married to Edith Wygath, of Denison, Iowa, who, with two children, Lucille and Gaylord, survives him.

His life was filled with fondness and devotion for his family. He was a man with great strength of character which, together with his fairness and integrity, caused him to be admired by all his personal and business associates. His early railroad experience gave him broad vision, and had much to do with the congenial atmosphere which surrounded all his work as a contractor.

He was a member and Trustee of the First Congregational Church of Springfield, Mo., and was connected with the Masonic Fraternity as a member of Solomon Lodge No. 271 and, also, Abou Ben Adam Shrine.

Mr. Weeks was elected an Associate Member of the American Society of Civil Engineers on June 24, 1914.

* Memoir prepared by H. B. Barry, Prin. Asst. Engr., St. L. S. F. Ry., St. Louis, Mo.

JOHN BAPTISTE LOBER, Affiliate, Am. Soc. C. E.*

DIED DECEMBER 21, 1924.

John Baptiste Lober was born in Camden, N. J., on April 11, 1848. He was educated in the public schools of Camden, and Pierce Business College, Philadelphia, Pa.

In 1870, he entered business with the firm of Warren Kirk and Company of Philadelphia, manufacturers of coal-tar products. Mr. Lober and Mr. E. Burgess Warren re-organized this Company and continued the business as the Warren-Lober Company until 1883. In that year the Warren-Lober Company was purchased by Mr. M. Ehret, backed by Messrs. W. L. Elkins and P. A. B. Widener, and the name was changed to the Warren Ehret Company, with Mr. Lober as President. Although organized as a roofing business, it soon became engaged in concrete work, particularly, sidewalks and floors. In view of the Company's success in this work, a consolidation proposal was offered by the Vulcanite Paving Company and accepted in 1890. Mr. Lober became Vice-President of the consolidation which retained the name of the Vulcanite Paving Company. This firm, one of the largest of its kind in the United States, used large quantities of imported German cement; at that time there was no Portland cement in the United States sufficiently uniform in character for use by the Vulcanite Paving Company which had established a reputation for superior quality of work.

Business increased to such an extent that it became advisable for the Company to manufacture its own cement. It acquired a tract of land in Warren County, New Jersey, containing a high grade of cement rock and, in 1894, its owners organized the Vulcanite Portland Cement Company, of which Mr. George W. Elkins became President, Dr. A. B. Bonneville, Vice-President, and Mr. Lober, Secretary and Treasurer. A cement mill was erected with two 40-ft. kilns and a daily capacity of 125 bbls. From the organization of the Company in 1894, Mr. Lober was its active Manager. He recognized the need for a domestic cement which would be equal in quality to the best foreign brand. The first 1 000 bbls. were rejected by Mr. Lober as not satisfactory, but, thereafter, the product met all the requirements of the paving companies and others engaged in similar business.

In 1903, Mr. Lober resigned as an officer of the Vulcanite Paving Company to become President of the Vulcanite Portland Cement Company, with which he served actively until a few months before his death at Wynnewood, Pa., on December 21, 1924.

Mr. Lober was a leader in the cement industry. He was one of the organizers of the Portland Cement Association in 1902. He served as a second President of the Association and as its President for nine years. He was Treasurer of the Eastern State Institution for the Feeble Minded and Epileptics at Spring City, Pa., and a member of the Union League Club of Philadelphia, and the American Society for Testing Materials.

* Memoir prepared by Albert Moyer, Affiliate, Am. Soc. C. E.

He had charge of large construction and manufacturing projects and was a highly talented, honorable, upright, and just business man. The memory of his achievements will always be an inspiration for others.

One of the last contributions which Mr. Lober made to the Portland cement industry was the book, "History of the Portland Cement Industry in the United States", written in collaboration with R. W. Lesley, Affiliate, Am. Soc. C. E., and Mr. George S. Bartlett. The work covered a period of nearly two years and included personal reminiscences, data, and historical facts relating to the development of the Portland cement industry in the United States.

Mr. Lober is survived by his wife, Mrs. Clara Lober, and a son, William D. Lober.

Mr. Lober was elected an Affiliate of the American Society of Civil Engineers on July 9, 1906.

KENNETH LOUIS GREEMAN, Jun. Am. Soc. C. E.*

DIED AUGUST 21, 1924.

Kenneth Louis Greeman, the son of Louis F. and Ida W. Greeman, was born in Batesville, Ind., on September 19, 1900. In his early childhood, his parents moved to Seymour, Ind., where he received his elementary education. He was graduated from the High School at Seymour in June, 1917, and entered Purdue University, West Lafayette, Ind., the following September, from which he was graduated four years later with the degree of Bachelor of Science in Civil Engineering.

While in college, Mr. Greeman spent his summer vacations in the pursuit of his chosen vocation, having been employed as Rodman with the Maintenance of Way Department of the Baltimore and Ohio Railroad in West Virginia; as Assistant Junior Engineer of road construction at Seymour, Ind., for the Indiana State Highway Commission; and as Assistant Superintendent of road construction, also at Seymour, for the Davis Construction Company.

After completing his college work, Mr. Greeman accepted a position as Assistant Engineer of Design with the Department of Bridge Construction of the Indiana State Highway Commission. With the exception of four months spent in the supervision of the construction, for the State, of a reinforced concrete arch bridge south of Orleans, Ind., designed by himself, Mr. Greeman continued in this position until May, 1923, when he resigned to accept a position as Designer with the Gabriel Steel Company, at Detroit, Mich.

In September, 1923, he received an appointment as an Instructor in the Department of Theoretical and Applied Mechanics at the University of Illinois, Urbana, Ill., which position he held until June, 1924. Although he had had no previous experience, he soon gained the confidence of his students and proved himself to have the instincts and ideals of a teacher by his interest and by thoroughness in his work, combined with his spirit of helpfulness and cooperation.

After having spent the summer in the employ of the Gabriel Steel Company, Mr. Greeman, with his family, was driving from Detroit to Seymour, Ind., where he had accepted an instructorship in the Department of Civil Engineering of Rose Polytechnic Institute. Their automobile was struck by a traction car near McCordsville, Ind., and Mr. Greeman and his infant son were instantly killed, while his wife was seriously injured.

In 1922, Mr. Greeman was married to Rita Watton, of Flint, Mich., and he is survived by his wife, his parents, and two brothers, Garnet A. and Omer W. Greeman.

He was a member of the First Methodist Episcopal Church of Champaign, Ill., and of the Tau Beta Pi and Pirathon Fraternities.

Mr. Greeman was elected a Junior of the American Society of Civil Engineers on January 16, 1922.

* Memoir prepared by E. R. Feldman, Esq., Auburn, Ind.

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